APPENDIX B

PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT PROPOSED PLUMPJACK SQUAW VALLEY INN ADDITION PLACER

KLEINFELDER

PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT
PROPOSED PLUMPJACK SQUAW VALLEY INN ADDITION
PLACER COUNTY, CALIFORNIA

December 27, 2000

December 27, 2000
File: 30-2583-02.001

K. B. Foster Civil Engineering, Inc.
P.O. Box 129
Carnelian Bay, California 96140

Attention: Ms. Suzanne Larson

**SUBJECT: Preliminary Geotechnical Investigation Report
Proposed PlumpJack Squaw Valley Inn Addition
Placer County, California**

Dear Ms. Larson:

The attached report presents the results of our geotechnical investigation for the proposed PlumpJack Squaw Valley Inn Addition which will be located west of the existing building along Squaw Peak Road in Squaw Valley, California. Our work consisted of subsurface exploration, laboratory testing, engineering analyses, and preparation of this report.

Based on our work completed to date, we have drawn the following general conclusions:

- Site soils encountered in our field investigation generally consisted of a surface layer of loose to medium dense granular fill over loose to dense silty sands. Several layers of very loose soils were encountered in localized areas.
- Groundwater was encountered during our exploration at approximately 23 feet below existing grade. During our previous investigation, groundwater was encountered at approximately 14 feet below grade, with indications of past groundwater levels of approximately 11 feet. Groundwater is not anticipated to affect the proposed underground parking structure provided that floor excavations extend no deeper than ten feet below existing grade. Excavations extending below 10 feet will require a subsurface perimeter drainage system to collect and direct water away from basement walls and foundations and waterproofing applied to the basement walls to limit groundwater infiltration during seasonal highs. Based on limited subsurface information, groundwater appears to have a northeast gradient.
- Site soils above ten feet are relatively dry and cohesionless. Excavations in these materials will need to be properly shored or sloped back to reduce caving and/or sloughing.

- Due to loose near subsurface conditions, the use of conventional shallow foundations for building support without prior ground modification will be unlikely. We have included preliminary recommendations for deep foundations (driven precast concrete piles or concrete filled steel piles) founded at approximately 35 feet below existing grade. However, other options including ground modification techniques such as vibroflotation columns should be considered once structural loads are determined.

These and other conclusions and recommendations, along with restrictions and limitations on these conclusions, are discussed in the attached report.

We appreciate this opportunity to be of service to you, and look forward to future endeavors. If you have any questions regarding this report or need additional information or services, please feel free to call one of the undersigned in our Reno office.

Sincerely,

KLEINFELDER, INC.

REGISTERED PROFESSIONAL ENGINEER
MARK J. DOEHRING
No. C 052213
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CIVIL
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Mark Doehring, P.E.
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Enclosures: Report (2 Bound, 1 Unbound)

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PRELIMINARY GEOTECHNICAL INVESTIGATION REPORT
PROPOSED PLUMPJACK SQUAW VALLEY INN ADDITION
PLACER COUNTY, NEVADA

1. INTRODUCTION AND SCOPE

1.1 Project Description

This report presents the results of our preliminary geotechnical study for the proposed PlumpJack Squaw Valley Inn Addition, which will be located west of the existing building along Squaw Peak Road in Squaw Valley, California. The site location is shown on the attached vicinity map (Plate 1). We understand that the project will include the expansion of the existing PlumpJack Squaw Valley Inn by constructing a new structure which will include 34 multi-family residential units (condominiums), underground parking and street level parking, mezzanine area and lobby. The structure will be up to six stories above existing ground level at its highest point, with the majority of the addition being five stories above grade. A lower parking level will be located approximately 8 to 14 feet below existing grade. The entire new structure will have a foot print of approximately 145,520 square feet. We understand the structure will be of reinforced concrete for the lower stories, with steel and wood frame construction for the upper stories. The bottom floor will be concrete slab-on-grade. Structural loads were not available at the time of the preparation of this report. Cuts for building pad construction are anticipated to exceed 14 feet.

1.2 Purpose and Scope of Work

The purpose of this study is to review our previous work in the site and further explore and evaluate the encountered subsurface conditions at the project site, and to provide our geotechnical recommendations and opinions for planning and preliminary design. Our geotechnical recommendations and opinions include:

- General soil and groundwater conditions at the project site, with emphasis on how the conditions are expected to affect the proposed construction;
- Suggested specifications for earthwork construction, including site preparation recommendations, a discussion of reuse of existing near surface soils as structural or non-structural fill, and a discussion of remedial earthwork recommendations, if warranted;
- Recommendations for temporary unconfined excavations;
- Recommended foundation types;
- Subgrade preparation for slab-on-grade concrete;
- Preliminary structural sections for asphalt concrete pavements; and
- Lateral earth pressures and drainage recommendations for subterranean structures, including single story basement and low height retaining walls.

It is understood that our study will be used by K.B. Foster Civil Engineering for the preliminary design of the subject project and submitted to the Placer County Planning Department.

Our scope of services consisted of background review, site reconnaissance, field exploration, laboratory testing, engineering analyses, and preparation of this report.

1.3 Authorization

Authorization to proceed with our work on this project was originally provided by Mr. Gavin Newsome of the PlumpJack Development Fund on July 17, 1997 in the form of a signed Kleinfelder Standard Form of Agreement. The scope of work was later updated, with authorization to proceed provided by Ken Foster of K. B. Foster Civil Engineering on November 14, 2000 in the form of a signed Kleinfelder Standard Form Agreement.

1.4 References

The following information was provided to Kleinfelder in the course of this study and serves as the basis of our understanding of the project type and scope.

- A preliminary 1"=40"-scale site plan sheet entitled *Topographic Survey PlumpJack Squaw Valley Inn, PlumpJack Development Fund*, prepared by K.B. Foster Civil

Engineering, Inc., dated May 2000. This drawing was the basis for the site plan shown on Plate 1 of this report.

- A preliminary 1"=30"-scale site plan sheet entitled *Topographic Survey PlumpJack Squaw Valley Inn, PlumpJack Development Fund*, prepared by K.B. Foster Civil Engineering, Inc., and dated October, 1996.

- Set of architecture drawings prepared by MacDonald Architects, *PlumpJack Squaw Valley Inn Addition*, Project 96.012, undated. The proposed building layout provided in the architecture drawings was overlaid onto the topographic survey sheet to complete the site plan shown on Plate 1.

In addition, the following published and unpublished references were reviewed during preparation of this report.

dePolo, C. M., J. G. Anderson, D. M. dePolo, and J. G. Price (1997), *Earthquake Occurrence in the Reno-Carson City Urban Corridor*, Seismological Research Letters, Volume 68, Number 3.

Hart, E. W., W. A. Bryant and T. C. Smith, *Summary of Faults Evaluated Program, 1983 Area, Sierra Nevada Region (Northern)*, Division Mines and Geology, State of California.

Geotechnical Investigation for Proposed PlumpJack Squaw Valley Inn Project Squaw Valley, California, Kleinfelder, Inc., August 14, 1986, File No. R-1665-1.

Jennings, C. W. (1994), *Fault Activity Map of California and Adjacent Areas*, California Geologic Date Map Series, Map No. 6, Division Mines and Geology, State of California.

Saucedo, G. J., D. L. Wagner (1992), *Geologic Map of the Chico Quadrangle, California, 1:250,000*, Division Mines and Geology, State of California.

2. METHODS OF STUDY

2.1 Field Exploration

Our selection of field exploration locations was based on the anticipated project layout and site access. The subsurface exploration consisted of drilling three borings in the proposed construction area using CME 55 truck mounted drill rig with hollow stem augers. Boring depths ranged from 30 to 41-1/2 feet below the existing ground surface. Locations of the borings shown on the Site Plan (Plate 1, Appendix A) were approximated by pacing from features shown on the topographic survey plan. These locations should be considered accurate only to the degree implied by the method used.

Soil conditions encountered are presented on the boring logs which are included as Plates 2 through 4. A description of the Unified Soil Classification System used to identify the site is presented on Plate 5 (Appendix A).

A field engineer logged the soil conditions exposed in the borings and collected bag and relatively undisturbed driven samples for laboratory testing. Bag samples were obtained by driving a 1-3/8-inch ID, 2-inch OD Standard Penetration Sampler into the bottom of the boring. Relatively undisturbed samples were collected by driving 2-inch ID, 2-1/2-inch OD Modified California Sampler, containing thin brass liners. The number of blows required to drive the last 12 inches of an 18-inch drive with a 140 pound hammer dropping 30 inches is recorded as the blows per foot (Blow Count) on the boring logs. When the sampler was withdrawn from the boring, the drive samples and brass liners containing the samples were removed, examined for logging, labeled and sealed to preserve the natural moisture content for laboratory testing. The results of this testing are presented on the boring logs. Due to the granular nature of the site soils no measurements of undrained shear strength using the torvane or penetrometer device were taken. After borings were completed, they were backfilled with excavated soil using the equipment at hand. Backfill was loosely placed and <u>not</u> compacted to the requirements typically specified for engineered fill.

Previous preliminary investigation had been carried out on the site by Kleinfelder, described in our report referenced in Section 1.4. The investigation included drilling seven borings up to 41-1/2 feet below the existing ground surface. Bore logs from this previous investigation are included in Appendix B for convenience. Approximate boring locations are shown on Plate 1 (Appendix A).

2.2 Laboratory Testing

Laboratory testing is useful for evaluating both index and engineering properties of soils. Typical index tests evaluate soil moisture content, unit weight, soil particle gradation, and plasticity characteristics. Tests for engineering properties can assess soil strength, compressibility, swell potential, and potential steel corrosion or adverse reactivity with Portland Cement Concrete. We performed laboratory testing on selected soil samples to assess the following:

- Soil Classification (ASTM D422 and D1140)
- Unit Weight and Moisture Content (ASTM D2937 and D2216)
- Direct Shear Strength (ASTM D3080)

In addition, the following analytical tests were performed by Chemax Laboratories:

- Soluble Sulfate Content
- Resistivity and pH

Individual laboratory test results can be found on the boring logs and on Plates 6 through 9, Appendix A, at the end of this report.

3. DISCUSSION

3.1 Site Conditions

The proposed addition is located just west of the existing PlumpJack Squaw Valley Inn along Squaw Peak Road. The existing improvements include a paved parking lot with landscaping, basketball courts and natural vegetation. Vegetation consists of grassed areas and numerous large pine trees up to 32 inches in diameter. The ground surface in the area of the proposed building slopes slightly to the northeast with a total relief of less than three feet within the proposed building area. The overall slope of the site is less than two percent. Drainage on the site consists of several drop inlets and a stormwater retention pond located northeast of the building area. Site stormwater is eventually piped into Squaw Creek to the north. It is known that fill soils were placed on the site to allow for parking during the 1960 Winter Olympics.

3.2 Regional Geology and Seismicity

The site is located near the center of the Sierra Nevada geomorphic province. The mountain ridges surrounding the site primarily consist of volcanic rocks of Quaternary to Tertiary age, with Cretaceous age granitic rocks forming the Sierra Crest west of the site. The volcanic rocks range from basaltic to andesitic in composition with the Tertiary-age andesite correlated with the Kate Peak Formation forming the basement rock underlying the site.

The spectacular features of the eastern Sierra Nevada mountains were formed by large scale faulting during Tertiary times. Most of the larger faults are no longer active, although many of the smaller faults in the area are still considered active or potentially active. Numerous volcanic vents are also located in the region. Although not currently active, these features have shown activity within the last 7 million years.

The project site is located at the western end of Squaw Valley on the valley floor. Much of the region has been affected by glaciation during the past 1.5 to 2 million years. This glacial activity is responsible for many of the landforms surrounding the site including the morainal deposits along the side of the valley.

The project site is located in UBC Seismic Zone 3. No potentially active or active faults have been mapped crossing or trending towards the project site. The nearest active fault is the northeast trending North Tahoe fault located approximately 8 miles east of the site and an unnamed northeast trending fault located approximately 12-1/2 miles northeast of the site. The North Tahoe fault is estimated to have a potential earthquake moment magnitude of 7.0. The unnamed earthquake fault ruptured in 1966 for an estimated length of 10 miles. The earthquake had an estimated Richter Magnitude of 6+. Should a seismic event occur, the site will most likely be effected by ground shaking; however, it is our opinion that the potential for fault-related surface rupture at the site is very low.

3.3 Subsurface Conditions

The following paragraphs summarize the results of our field exploration. The boring logs should be reviewed for a more detailed description of the subsurface conditions at the locations explored.

Near surface soils consisted of loose to medium dense, relatively clean sand and gravel extending to depths of approximately 10 to 16 feet below existing grade. This unit is believed to be fill material placed just prior to the 1960 Winter Olympics. The original ground surface appears to have been stripped of vegetation prior to fill placement, since no topsoil or vegetation was observed in any of our borings at the fill/native soil interface.

Underlying the fill, we encountered native soils of loose to medium dense silty sand becoming medium dense to dense below approximately 20 to 30 feet. In boring B3, the native silty sands were loose to medium dense below approximately 22 feet. An unusually very loose layer of silty sand was encountered in boring B2 at approximately 15 to 23 feet. This unit appears to contain a significant proportion of fine-grained material.

Groundwater was encountered during our exploration at depths ranging from 22-1/2 to 23 feet below the ground surface. At the time of our previous investigation, groundwater was encountered at depths of approximately 14 to 16 feet below the existing surface. Changes in the soil color, such as orange or rust, above the existing water level are typically an indication of past groundwater levels. It is our opinion that the periodic high groundwater level could potentially rise within 10 feet of the existing ground surface. Based on limited information, groundwater appears to have a northeast gradient. Fluctuations in the level of the groundwater and soil moisture conditions as noted in this report may occur due to variations in precipitation, snow melt, and other factors.

3.4 Laboratory Test Results

Laboratory testing was performed as previously discussed in Section 2.2. The test data were evaluated in combination with our field exploration information to assess the engineering properties of the predominant soil types. The laboratory test data revealed that site soils generally consist of granular materials with excellent steel corrosion resistance and negligible sulfate reaction with Portland cement concrete.

3.5 Analytical Methods

Field and laboratory data are useful when combined with engineering fundamentals to assess specific behavior such as bearing capacity, settlement, slope stability, and other design parameters. The following approaches were used in developing the conclusions and recommendations presented in subsequent sections of this report.

- Allowable bearing pressures, estimated settlements and lateral load capacities for pile design were calculated using methods from NAVFAC DM-7.2, September 1986.

- Lateral earth pressures were developed using Rankin's approach for passive and active resistance.

- Pavement sections were developed using the California Gravel Equivalent Method. This method uses as its basis the total traffic expected for the project and the subgrade soil strength evaluated using an assumed R-value.

4. CONCLUSIONS

The following conclusions are based on the data collected during this assessment and are subject to the limitations stated in this report. These conclusions may change if additional information becomes available. Based on the results of our study, no severe soil or groundwater constraints were observed which would preclude development. The following is a summary of our conclusions.

- Site soils encountered in our field investigation generally consisted of a surface layer of loose to medium dense granular fill over loose to dense silty sands. Several layers of very loose soils were encountered in localized areas.
- Groundwater was encountered during our exploration at approximately 23 feet below existing grade. During our previous investigation, groundwater was encountered at approximately 14 feet below grade, with indications of past groundwater levels of approximately 11 feet. Groundwater is not anticipated to affect the proposed underground parking structure provided that floor excavations extend no deeper than ten feet below existing grade. Excavations extending below 10 feet will require a subsurface perimeter drainage system to collect and direct water away from basement walls and foundations and waterproofing applied to the basement walls to limit groundwater infiltration during seasonal highs. Based on limited information, groundwater appears to have a northeast gradient.
- Site soils above ten feet are relatively dry and cohesionless. Excavations in these materials will need to be properly shored or sloped back to reduce caving and/or sloughing.
- Due to loose near subsurface conditions, the use of conventional shallow foundations for building support without prior ground modification will be unlikely. We have included preliminary recommendations for deep foundations (driven precast concrete piles or concrete filled steel piles) founded at approximately 35 feet below existing grade. However, other options including ground modification techniques such as vibroflotation columns should be considered once structural loads are determined.

Specific recommendations for project design and construction including mitigation of potential problems described above are presented in Section 5.0.

5. RECOMMENDATIONS

5.1 Site Clearing and Preparation

Prior to construction, surface vegetation and organic soils should be stripped and removed from the site or stockpiled for use in non-structural areas. It appears four inches can be used as a reasonable estimate for average depth of stripping. Deeper stripping/grubbing of organic soils, tree roots, etc., will be required in localized areas. Tree root balls should be removed and the resulting voids backfilled with adequately compacted backfill soil. Existing pavements should be demolished and removed from the site. The geotechnical engineer should be present during demolition, stripping and site preparation operations to observe excavation, stripping and grubbing depths, and to evaluate whether buried obstacles such as underground utilities are present. Special care should be exercised in evaluating whether loose utility backfills exist which could adversely affect the planned structures. Excavations resulting from removal operations should be cleaned of all loose material and widened as necessary to permit access to compaction equipment.

Dust control will be the responsibility of the contractor. A dust control plan should be prepared by the owner, civil engineer, or contractor prior to the start of grading.

5.2 Earthwork

5.2.1 General Site Grading

Site preparation and grading should conform to the requirements contained in this report and in the suggested specifications, which are provided as Appendix C of this report. We anticipate that site grading can be performed with conventional earthmoving equipment. Although not encountered in our current investigation, it has been our past experience that soils directly underlying pavements typically become saturated, as the impermeable pavements prevent capillary water from evaporating. Therefore, several feet of wet, yielding subgrade soils may be encountered in areas of existing pavements. If fill is necessary, prior to fill placement, the exposed native soils should be scarified to a minimum depth of six inches, moisture conditioned as necessary, and compacted to a minimum of 90% relative compaction in accordance with the ASTM D1557 compaction test method.

Where fill is necessary, materials should meet the gradation and plasticity requirements listed for "structural fill" in Appendix C. It appears that the existing site soils should generally be capable of meeting recommended requirements for structural fill. Fill placement and compaction requirements presented in Appendix C should be followed.

5.2.2 Temporary Unconfined Excavations

We understand that deep cuts of up to 14 feet are proposed to construct the underground parking. We recommend that open slope excavations for parking be excavated at a maximum short-term allowable slope of 1-1/2:1 H:V. Excavations should be performed during the summer months to limit the potential of encountering seasonal high groundwater levels.

The above suggested layback is a guideline which may require modification in the field after the start of construction. The contractor is ultimately responsible for the safety of workers and should strictly observe federal and local OSHA requirements for excavation shoring and safety. Due to the granular nature of the surface soils, some ravelling of temporary cut slopes should be anticipated. During wet weather, runoff water should be prevented from entering excavations.

Excavations for underground parking should be performed during the summer months in order to minimize the potential of spring runoff from affecting the excavation.

5.2.3 Temporary Trench Excavation and Backfill

It appears that excavations for footings and utility trenches can be readily made with either a conventional backhoe or excavator in either native soil or compacted imported fill. Sloughing of trench walls will likely occur in the near surface cohesionless fill materials. Excavations should be evaluated to verify their stability prior to occupation by construction personnel. Shoring or sloping of trench walls may be necessary to protect personnel and provide temporary stability. All excavations should comply with current OSHA safety requirements for Type C soils. (Federal Register 29 CFR, Part 1926).

During wet weather, runoff water should be prevented from entering excavations. Water should be collected and disposed of outside the construction limits. Heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a distance of one-third the slope height from the top of any excavation.

Backfills for trenches or other excavations within pavement areas, beneath slabs, and adjacent to foundations should be compacted in six- to eight-inch layers with mechanical tampers. Jetting and flooding should not be permitted. We recommend all backfill be compacted to a minimum compaction of 90% of the maximum dry density as determined by ASTM D1557. The moisture content of compacted backfill soils should be within 2% of the optimum. Poor compaction in utility trench backfill may cause excessive settlements resulting in damage to the pavement structural section or other overlying improvements. Compaction of trench backfill outside of improvement areas should be a minimum of 85% relative compaction.

5.3 Foundation Design Parameters

5.3.1 General

The site is located in UBC Seismic Zone 3. If siesmic loadings are evaluated using the 1997 UBC method, we recommend using a seismic zone factor of 0.3 and a soil profile type S_D. The S_D soil profile type is applicable to stiff soil conditions within the top 100 feet of the soil profile, with a minimum shear wave velocity of 600 feet/second, as outlined in Table No. 16-J of the UBC.

Consideration has initially been given to the use of shallow footings, but their use is not recommended due to loose near surface soil conditions. It is possible that shallow foundations may be applicable if ground modification techniques such as vibroflotation columns are used; however, foundation design used in conjunction with ground modification techniques is beyond our current scope of work.

Uncased bored piers would also not be suitable on this site, since caving conditions will preclude satisfactory construction. Without final structural load information, for preliminary design, we recommend the proposed structure be supported on driven precast concrete or concrete filled steel pipe piles extending to a depth of approximately 35 feet below existing grade. For budgeting purposes only, we recommend using a pile length of 40 feet for production piles. Allowable axial capacities, estimated settlements, and specific design and construction recommendations for driven precast concrete and concrete filled steel pipe piles of various sizes are provided below.

Note: Axial capacities and estimated settlements given below assume that all recommendations provided are implemented during design and construction of the pile foundation system. If pile load tests are not performed, recommended axial capacities should be reduced by 10 percent.

5.3.2 Allowable Axial Capacities

Allowable axial capacities for various pile sizes and load conditions are presented in the table below.

Pile Size (inches)	X-Sectional Shape	Estimated Tip Elevation (below existing grade)	Allowable Axial Capacities (kips)	
			Downward	Upward
14 x 14	Square	35 feet	120	45
16 x 16	Square	35 feet	160	35
20 x 20	Square	35 feet	260	70
24 x 24	Square	35 feet	360	90
12-3/4	Circular	35 feet	65	25
16	Circular	35 feet	115	35

Notes regarding table above:

1. Provided capacities include the weight of pile.
2. Provided downward capacities (dead plus live loads) may be increased by 1/3 for short-term loads due to wind or seismic loads.
3. No reduction in the axial capacity of an individual pile is necessary within pile groups provided a center-to-center spacing of at least 3 diameters is used.

5.3.3 Estimated Settlement

Maximum settlement of an individual pile design and constructed in accordance with the recommendations provided herein is expected to be less than 3/4 inch, not including elastic compression of the pile under design loads.

5.3.4 Pile-to-Pile Connection

Depending on applicable building code requirements, the lowest level floor slab may be used as a lateral load-carrying connection between pile caps and in lieu of grade beams. If used, the slab should be designed by the project Structural Engineer to withstand anticipated, imposed loads.

5.3.5 Indicator Pile Program

Before production piles are cast, we recommend six indicator piles be driven at representative locations in order to verify estimated pile lengths. Indicator piles should be driven to a minimum depth of 40 feet below existing site grade. We recommend indicator piles be cast at least five feet longer than the anticipated production pile length. Selected indicator piles should be redriven one day following initial installation in order to evaluate post-construction pile set-up and strength gain.

5.3.6 Heave Monitoring

All individual piles within a pile group should be monitored for vertical heave during driving of adjacent piles. We recommend the pile top elevation be determined immediately upon completion of driving of each pile and checked after all piles within the group have been installed. Heaved piles should be redriven to the design driving criteria.

5.3.7 Condition Survey

We recommend a condition survey of adjacent buildings be performed prior to driving indicator piles. As a minimum, the condition survey should include photographs of the exterior of adjacent structures and the placement and periodic measurement of vertical and horizontal survey monuments affixed to these structures.

5.3.8 Vibration Monitoring

Piles driven within 50 feet (measured horizontally) of existing structures may result in vibration-induced distress to these structures. Prior to pile driving, we recommend existing structures be evaluated by a structural engineer to establish a limiting response spectra. Further, we recommend the indicator pile program include driving at least one indicator pile at the closest point of approach to adjacent structures.

During driving, existing structures should be monitored for driving-induced displacements, velocities, and accelerations by a firm qualified in vibration monitoring. If during indicator pile driving the limiting response spectra established by the structural engineer is exceeded, driving should be modified to reduce vibrations below acceptable limits. Modifications to pile driving could include pre-drilling, reducing the energy output of the hammer, and/or mobilizing a pile driver with a lower energy rating.

5.4 Concrete Slab-on-Grade Construction

We recommend that site concrete slabs be supported on a minimum 12 inch thick fill mat consisting of suitable on-site or imported fill which has been compacted to a minimum of 95% relative compaction. Subgrade soils beneath the underground parking may be recompacted in-place.

All concrete floor slabs should have a minimum thickness of four inches. Slab thickness and structural reinforcing requirements within the slab should be determined by the design engineer. At least four inches of Type 2 aggregate base should be placed beneath slab-on-grade floors to provide uniform support. The aggregate base should be compacted to a minimum of 95% relative compaction. Uplift resistance may need to be considered during design depending on the final finished floor elevation.

We recommend that the base course be placed within three to five days (depending on the time of year) after moisture conditioning and compaction of the subgrade soil. The subgrade should be protected against drying until the concrete slab is placed.

In floor slab areas where moisture sensitive floor coverings are planned, an impermeable membrane (e.g. 10-mil thick polyethylene) should be placed over the base course to reduce the migration of moisture vapor through the concrete slabs. The impermeable membrane should be protected by two inches of fine, moist sand placed both above and below the membrane. The sand cover will provide protection for the membrane and will promote uniform curing of the concrete slab. The sand cover should be moistened and tamped prior to slab placement.

5.5 Retaining Structures

Lateral earth pressures will be imposed on all subterranean structures, including basements and retaining walls. Table 1 presents a list of soil parameters which we recommend for design of these structures assuming a level backfill.

TABLE 1

LATERAL EARTH PRESSURES

Earth Pressure	Equivalent Fluid Density
Active	30 pcf
At-rest	45 pcf
Passive	375 pcf
Friction Coefficient	0.45

The at-rest case is applicable for braced walls where rotational movement is confined to less than 0.001 H. If greater movement is possible, the active case applies. These values are for horizontal backfill and do not include hydrostatic pressures that might be caused by groundwater or surface water trapped behind a structure. Where backfill is placed against structures such as basement walls, we recommend that non-expansive, free-draining materials meeting filter criteria be used in the zone immediately adjacent to the structure to reduce hydrostatic forces. Alternately, the use of pre-manufactured drainage panels should be considered. Furthermore, adequate drainage of the backfill in the form of subdrains and/or weepholes should be provided at the base of the wall.

The lateral loads computed using the values in Table 1 assume that the non-expansive backfill will extend laterally at least one-half of the wall height. If this condition does not apply, the design values may require revision. This backfill should be compacted to 90% of maximum dry density and within 2% of the optimum moisture content as determined by ASTM D1557. Over-compaction should be avoided as the increased compactive effort will result in lateral pressures higher than those recommended above. Heavy compaction equipment or other loads should not be allowed in close proximity to the wall unless planned for in the structural design.

5.6 Pavement Sections

The pavement sections provided in our previous report have been reviewed and appear to be acceptable for the estimated traffic loadings and frequencies. The recommended pavement structural sections and estimated traffic loadings for the project are presented in Table 2.

A Traffic Index of 4.5 corresponds to light traffic loadings and frequencies, such as for automobile parking areas. A Traffic Index of 6 is generally used for moderate traffic loadings and frequencies, such as main entrance and exit ways and delivery truck corridors. A minimum R-Value of 75 was used for aggregate base in our design.

TABLE 2

PAVEMENT STRUCTURAL SECTIONS

Traffic Index	Recommended Minimum Structural Section
4.5	2.5″ of asphalt concrete on 4″ of aggregate base
6	3.5″ of asphalt concrete on 4″ of aggregate base

Placement and compaction procedures for materials and construction should conform to the suggested specifications contained in Appendix C of this report. The sections presented in Table 2 are based on a R-Value test performed on a selected sample obtained during our initial investigation and should be considered preliminary in nature. We recommend verification of soil conditions as construction progresses so that appropriate revisions can be made if necessary.

The pavement structural sections presented in Table 2 are designed for the assumed traffic loadings. However, based on our experience in the Tahoe area, environmental aspects such as freeze-thaw cycles and thermal cracking will probably govern the life of AC pavements. Thermal cracking of the asphalt pavements allows more water to enter the pavement section which promotes deterioration and increases maintenance costs. To reduce the long-term pavement maintenance costs, we recommend a minimum AC section of three inches asphalt concrete on four inches of aggregate base be considered for low traffic areas.

It should be noted that the subgrade soils are likely to be prone to frost action during the winter and saturation during the wet spring months. The primary impact of frost action and subgrade saturation is the loss of subgrade and aggregate base strength. Pavement life will be increased if efforts are made to reduce the accumulation of excess moisture in the subgrade soils. Consideration should be given to installing subdrainage in the form of trench drains in low areas, which are daylighted or tied to the storm drain system.

5.7 Site Drainage

Final elevations at the site should be planned so that drainage is directed away from all foundations. Parking areas should be sloped and drainage gradients maintained to carry all surface water off the site. In parking lot areas, curbs adjacent to landscaping should be deepened to act as a cutoff, or a subdrain system should be constructed to collect excessive water from landscaping irrigation. A wall and foundation drain is recommended to intercept snowmelt and direct it away from the structure.

5.8 Steel and Concrete Reactivity

Analytical testing of selected soil samples was performed to assess the potential for adverse reactivity with concrete and corrosivity with steel. A soluble sulfate test was performed to evaluate potential sulfate attack against Portland Cement Concrete. The soluble sulfate content was observed to be less than 10 ppm. Therefore, the potential for sulfate attack appears to be negligible and conventional Type I/Type II cement may be used according to data furnished by Cement Industry Technical Committee of California and Chemax Laboratories.

Resistivity tests are used as an indication of possible corrosion activity. Generally, the lower the native resistivity of the soils, the more likely that galvanic currents may occur and corrosion result. A resistivity test was performed on one select sample. Resistivity values for the near-surface native soils are on the order of 14500 ohm-cm and, therefore, appear to have a low corrosion potential where metal will be in contact with native soils.

6. ADDITIONAL SERVICES

6.1 Project Bid Documents

It has been our experience during the bidding process, that contractors often contact us to discuss the geotechnical aspects of the project. Informal contacts between Kleinfelder and an individual contractor could result in incorrect or incomplete information being provided to the contractor. Therefore, we recommend a pre-bid meeting be held to answer any questions about the report prior to submittal of bids. If this is not possible, questions or clarifications regarding this report should be directed to the project Owner or his designated representative. After consultation with Kleinfelder, the project Owner (or his representative) should provide clarifications or additional information to all contractors bidding the job.

6.2 Construction Observation/Testing and Plan Review

The recommendations made in this report are based on the assumption that an adequate program of tests and observations will be made during construction to verify compliance with these recommendations. These tests and observations should include, but not necessarily be limited to, the following:

- Observations and testing during site preparation and earthwork.
- Observation of footing trench excavations.
- Observation and testing of construction materials.
- Consultation as may be required during construction.

We also recommend that project plans and specifications be reviewed by us to verify compatibility with our conclusions and recommendations. Additional information concerning the scope and cost of these services can be obtained from our office.

The review of plans and specifications and the field observation and testing by Kleinfelder are an integral part of the conclusions and recommendations made in this report. If we are not retained for these services, the Client agrees to assume Kleinfelder's responsibility for any potential claims that may arise during construction.

7. LIMITATIONS

Recommendations contained in this report are based on our field explorations, laboratory tests, and our understanding of the proposed construction. The study was performed using a mutually agreed upon scope of work. It is our opinion that this study was a cost-effective method to evaluate the subject site and evaluate some of the potential geotechnical concerns. More detailed, focused, and/or thorough investigations can be conducted. Further studies will tend to increase the level of assurance, however, such efforts will result in increased costs. If the Client wishes to reduce the uncertainties beyond the level associated with this study, Kleinfelder should be contacted for additional consultation.

The soils data used in the preparation of this report were obtained from borings made for this investigation. It is possible that variations in soils exist between the points explored. The nature and extent of soil variations may not be evident until construction occurs. If any soil conditions are encountered at this site which are different from those described in this report, our firm should be immediately notified so that we may make any necessary revisions to our recommendations. In addition, if the scope of the proposed project, locations of structures, or building loads change from the description given in this report, our firm should be notified.

This report has been prepared for design purposes for specific application to the PlumpJack Squaw Valley Inn addition in accordance with the generally accepted standards of practice at the time the report was written. No warranty, express or implied, is made.

This report may be used only by the Client and for the purposes stated, within a reasonable time from its issuance. Land use, site conditions (both on- and off-site), or other factors including advances in man's understanding of applied science may change over time and could materially affect our findings. Therefore, this report should not be relied upon after 36 months from its issue. Kleinfelder should be notified if the project is delayed by more than 24 months from the date of this report so that a review of site conditions can be made, and recommendations revised if appropriate.

It is the CLIENT'S responsibility to see that all parties to the project including the designer, contractor, subcontractors, etc., are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the Contractor's option and risk. Any party other than the Client who wishes to use this report shall notify Kleinfelder of such intended use by executing the "Application for Authorization to Use" which follows this document as Appendix D. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the Client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.

APPENDIX A

Plates

Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	Sample	USCS	SOIL DESCRIPTION
		12				FILL: LIGHT BROWN GRAVELLY SAND (SM) slightly moist, medium dense, fine to coarse grained sand, fine gravel.
		12	9			FILL: LIGHT BROWN GRAVELLY SAND (SP) slightly moist, medium dense, fine grained sand, fine to coarse gravel, some silt.
		9				RED BROWN SILTY SAND (SM) moist, loose, very fine grained sand.
		8				Fine to coarse sand below 12 feet.
100	12.2	11	15			1/2" thick black/organic seam at approximately 15 feet.
		16				Same as above, sampler hit cobble.
		31				MEDIUM BROWN GRAVELLY SAND (SP) wet, medium dense to dense, fine to coarse sand and gravel.
		42				Approximately 1 foot of heave.

DEPTH IN FEET: 0, 2, 4, 6, 8, 10, 12, 14, 16, 18, 20, 22, 24, 26, 28, 30, 32, 34, 36, 38, 40, 42, 44, 46, 48, 50

DATE DRILLED: 8-5-97
TOTAL DEPTH: 31.50 feet

LOGGED BY: K.C. Wuelfing
EQUIPMENT: CME 55 Hollow Stem Auger Rig

KLEINFELDER

PROJECT NO. 30-2583-01.001

SQUAW VALLEY INN ADDITION
SQUAW PEAK ROAD
SQUAW VALLEY, CALIFORNIA
LOG OF BORING B-1

PLATE
2

DEPTH IN FEET	Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	Sample	USCS	SOIL DESCRIPTION
0							FILL: RED BROWN SILTY SAND (SM) slightly moist to moist, loose, fine to coarse grained sand, trace gravel.
2							
4			5				
6		17.1	5	29			Thin organic/black layer at approximately 6.5 feet.
8							
10			29				
12			15				Becoming moist at 11 feet, medium dense to dense. Only 3" on sample recovered.
14							RED GRAY SANDY GRAVEL (GP) slightly moist, medium dense, fine to coarse sand and gravel.
16		33.3	3	39			RED BROWN SILTY SAND (SM) moist to wet, very loose, fine to coarse grained sand, trace fine gravel.
18							
20			3				No sample recovered.
22		▽					
24							RED GRAY SANDY GRAVEL (GP) wet, medium dense, fine to coarse sand and gravel.
26			16				
28							
30			32				Becoming dense, pocket of clean sand at 30.5, becoming yellow brown at 31 feet.
32							
34							
36							SILTY GRAVELLY SAND (SM) wet, very dense, fine to coarse sand and gravel.
38							
40			54				
42							
44							
46							
48							
50							

DATE DRILLED: 8-5-97
TOTAL DEPTH: 41.50 feet

LOGGED BY: K.C. Wuelfing
EQUIPMENT: CME 55 Hollow Stem Auger Rig

KLEINFELDER

PROJECT NO. 30-2583-01.001

SQUAW VALLEY INN ADDITION
SQUAW PEAK ROAD
SQUAW VALLEY, CALIFORNIA
LOG OF BORING B-2

PLATE
3

Dry Density lb/ft3	Moisture Content %	Blows/ Ft.	Percent Passing #200	USCS	SOIL DESCRIPTION
					FILL: RED GRAY GRAVELLY SAND (SP) slightly moist, medium dense, fine to coarse sand and gravel, some silt.
		22			
	8.3	14	10		Becoming moist at 6 feet.
		28			Medium dense to dense.
		20			Very fine silty sand layer at 13 feet.
		12			DARK BROWN SILTY SAND WITH GRAVEL (SM) moist to wet, loose to medium dense, fine to coarse sand and gravel.
110	12.7				
		28			YELLOW BROWN SILTY GRAVELLY SAND (SM) wet, medium dense to dense, fine to coarse sand and gravel.
	▽				
		11	9		RED BROWN SAND (SW) wet, loose to medium dense, very fine to coarse sand, trace fine gravel, some silt.
	26.0				
		11			YELLOW BROWN SILTY SAND (SM) wet, loose to medium dense, very fine to medium grained sand.

DEPTH IN FEET

DATE DRILLED: 8-5-97
TOTAL DEPTH: 31.50 feet

LOGGED BY: K.C. Wuelfing
EQUIPMENT: CME 55 Hollow Stem Auger Rig

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PROJECT NO. 30-2583-01.001

SQUAW VALLEY INN ADDITION
SQUAW PEAK ROAD
SQUAW VALLEY, CALIFORNIA
LOG OF BORING B-3

PLATE 4

THE UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS				GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOIL More than 50% of the material is LARGER than the No. 200 sieve.	GRAVELS More than 50% of coarse part is LARGER than the No. 4 Sieve.	CLEAN GRAVELS Less than 5% finer than No. 200 Sieve.	PI<4	GW	Well graded gravels, gravel – sand mixtures, little or no fines, Cu>4 & 1<Cc>3
			PI>7	GP	Poorly graded gravels or gravel – sand mixtures, little or no fines Cu<4 or 1>Cc<3
		GRAVEL More than 12% finer than No. 200 Sieve.		GM	Silty gravels, gravel – sand – silt mixtures
				GC	Clayey gravels, gravel – sand – clay mixtures
	SANDS More than 50% of coarse part is SMALLER than the No. 4 Sieve.	CLEAN SANDS Less than 5% finer than No. 200 Sieve.		SW	Well graded sands, gravelly sands, little or no or no fines, Cu>6 & 1<Cc>3
				SP	Poorly graded sands or gravelly sands, little or no fines Cu<6 or 1>Cc<3
		SAND More than 12% finer than No. 200 Sieve.	PI<4	SM	Silty sands, sand – silt mixtures
			PI>7	SC	Clayey sands, sand – clay mixtures
FINE GRAINED SOIL More than 50% of the material is SMALLER than the No. 200 sieve.	SILTS AND CLAYS Liquid limit LESS than 50		PI–Below A–Line	ML	Inorganic silts, rock flour, or clayey silts of low plasticity
			PI–Above A–Line	CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
				OL	Organic silts & organic clays of low plasticity
	SILTS AND CLAYS Liquid limit GREATER than 50		PI–Below A–Line	MH	Inorganic silts, clayey silts, or silts of high plasticity
			PI–Above A–Line	CH	Inorganic clays of high plasticity, fat clays
				OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS				PT	Peat & other highly organic soils

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

PARTICLE SIZE LIMITS

BOULDERS	COBBLES	GRAVEL		SAND			SILT	CLAY
		Coarse	Fine	Coarse	Medium	Fine		

12" 3" 3/4" #4 #10 #40 #200 0.002 mm

DESCRIPTIVE TERMS USED WITH SOILS

CONSISTENCY & APPARENT DENSITY

	SILTS & CLAYS	SANDS & GRAVELS
Strongest	Hard	Very Dense
	Very Stiff	Dense
	Stiff	Medium Dense
	Medium Stiff	Loose
Weakest	Soft	Very Loose
	Very Soft	

MOISTURE CONTENT

Wettest	Wet
	Very Moist
	Moist
	Slightly Moist
Driest	Dry

∇ – Water Level Observed During Exploration

▼ – Water Level Observed After Exploration

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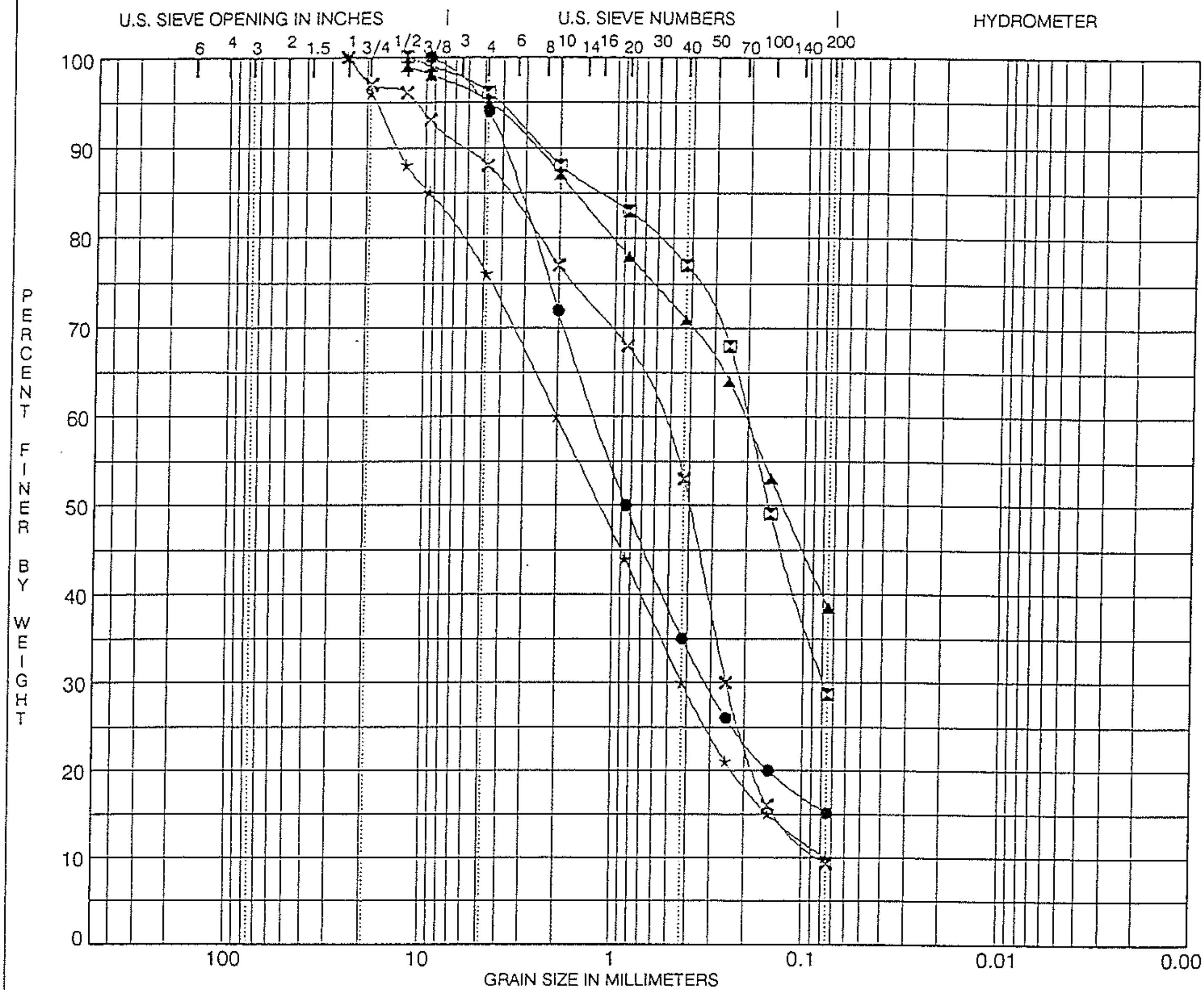
KEY TO SOIL CLASSIFICATION AND TERMS

SQUAW VALLEY INN ADDITION
SQUAW PEAK ROAD
SQUAW VALLEY, CALIFORNIA

PLATE 5

PROJECT NO. 30-2583-01.001

CAD FILE:



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring	Depth (ft.)	Description - ASTM Classification	MC%	LL	PL	PI	Cc	Cu
●	B-1	at 16.0	RED BROWN SILTY SAND (SM)	12					
⊠	B-2	at 5.0	RED BROWN SILTY SAND (SM)	17					
▲	B-2	at 15.0	RED GRAY SILTY SAND (SM)	33					
★	B-3	at 5.0	FILL: RED GRAY GRAVELLY SAND (SP)	8				1.20	26.7
✕	B-3	at 25.0	RED BROWN SAND (SW)					1.32	7.3

	Boring	Depth (ft.)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-1	at 16.0	9.50	1.25	0.317		6.0	78.9	15.1	
⊠	B-2	at 5.0	12.50	0.20	0.078		4.0	67.3	28.7	
▲	B-2	at 15.0		0.21				56.5	38.5	
★	B-3	at 5.0	25.00	2.00	0.425	0.0750	24.0	66.0	10.0	
✕	B-3	at 25.0	25.00	0.59	0.250	0.0806	12.0	78.7	9.3	

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SQUAW VALLEY INN ADDITION
SQUAW PEAK ROAD
SQUAW VALLEY, CALIFORNIA

PLATE 6

PROJECT NUMBER: 30-2583-01.001 August 1997

GRAIN SIZE ANALYSES

POTENTIAL REACTIVITY OF SOLUBLE SULFATES IN SOIL OR GROUNDWATER WITH PORTLAND CEMENT CONCRETE

TABLE 1: RECOMMENDATIONS FOR CONCRETE IN SULFATE ENVIRONMENTS *

Soluble Sulfates In Soil %	Sulfates In Water P.P.M.	Cement Type	Maximum Water/Cement Ratio	Minimum Cement Content — Lbs.
0-0.02	0-150	(Negligible	(Sulfate)	(Reaction)
0.02-0.10	150-1000	I or II	0.55	470
0.10-0.20	1000-2000	II	0.50	560
0.20-1.50	2000-15,000	II	0.45	660
		V	0.50	560
Over 1.50	Over 15,000	V	0.45	660

* NOTE A. Concrete for piling and other concrete in sea water environments may contain Type II cement when the water-cement ratio is a maximum of 0.50 or the cement factor is a minimum of 560 pounds. The sulfate concentration in Table I should govern in all cases.

* NOTE B. Sewage treatment facilities normally are constructed using Type II cement except in areas where high sulfate soils of waters exist (See Table I). In sewage, sulfides rather than where sulfates are formed. The sulfide combining with water in the presence of oxygen, can produce sulfuric acid to which no Portland cement is time resistant. Under these conditions, plastic liners, or coatings, are generally used. Closed tanks normally contain an atmosphere of methane rather than oxygen, so acid attack would not be likely to occur. Good quality concretes containing Type II cement with a maximum water cement ratio of 0.53 have provided excellent service in Los Angeles City and County sanitary treatment facilities.

Under special conditions, a concrete materials engineer should be consulted.

Reference: "Recommended Practice to Minimuze Attack on Concrete by Sulfate Soils and Water" by Cement Industry Technical Committee of California.

SAMPLE IDENTIFICATION	B-1 @ 10-11.5'			
SAMPLE DESCRIPTION	GRAVELLY SAND (SP)			
SOLUBLE SULFATE (%)	<0.001			
SOLUBLE SULFATES (PPM)	--			
COMMENTS	NEGLIGIBLE SULFATE REACTION			

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POTENTIAL REACTIVITY

SQUAW VALLEY INN ADDITION

SQUAW PEAK ROAD
SQUAW VALLEY, CALIFORNIA

PLATE 7

CAD FILE: A:\AUG_97\2583LAB.DWG

STEEL CORROSION POTENTIAL OF SOILS*

Corrosion Resistance	Resistivity (ohm-cm)
Excellent	6,000 to 10,000
Good	4,500 to 6,000
Fair	2,000 to 4,500
Bad	0 to 2,000

LABORATORY TEST RESULTS

Soil Type	Source	Resistivity (ohm-cm)	pH**
GRAVELLY SAND (SP)	B-1 @ 10-11.5'	14,540	5.80

* Reference: "Accelerated Corrosion Tests for Buried Metal Structures", by Paul Lieberman, Ph.D., in Pipeline and Gas Journal October, 1996, Pg.51

** Note: Corrosion potential of soils generally increases as pH decreases below 7.

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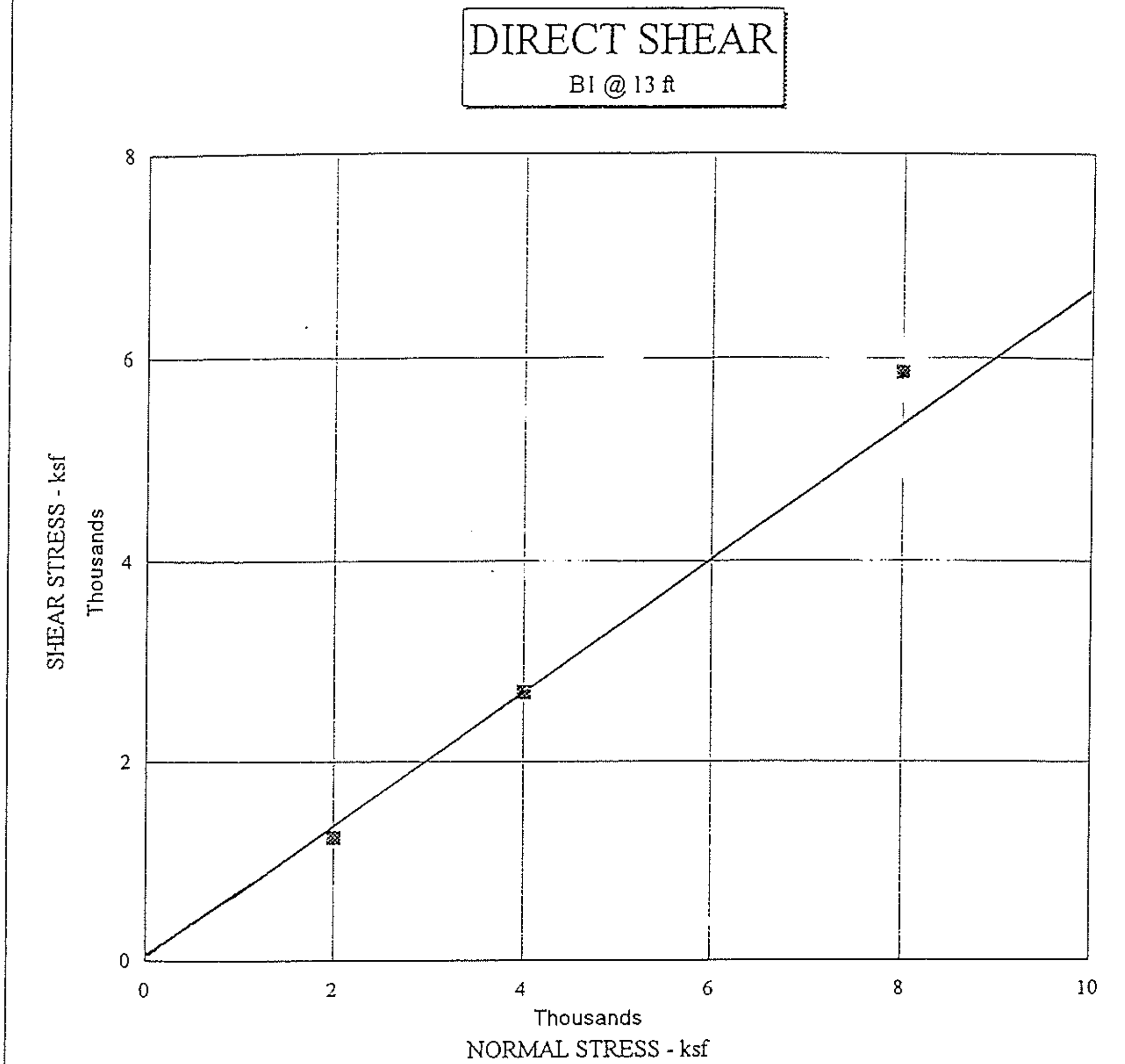
STEEL CORROSION POTENTIAL OF SOILS

SQUAW VALLEY INN ADDITION
SQUAW PEAK ROAD
SQUAW VALLEY, CALIFORNIA

PLATE 8

CAD FILE: A:\AUG_97\2583LAB.DWG



TEST TYPE:	CD/WET/STAGED
BORING NO:	B-1
DEPTH:	13 ft
SOIL DESCRIPTION:	Red Brown Silty Sand (SM)
RATE OF SHEAR:	0.0017 in/min

FRICTION ANGLE:	34 deg.
COHESION:	0 ksf

DRY DENSITY - pcf	113.7		
INITIAL WATER CONTENT - %	17.0		
FINAL WATER CONTENT - %	13.5		
NORMAL STRESS - ksf	2000	4000	8000
MAXIMUM STRESS - ksf	1234	2686	5857

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SQUAW VALLEY INN ADDITION
SQUAW PEAK ROAD
SQUAW VALLEY, CALIFORNIA

PLATE
9

PROJECT NO. 30-2583-01

APPENDIX B

Boring Logs
From Previous Geotechnical Investigation

LOG OF: Boring 1

EQUIPMENT: CME 55 Auger Drill Rig

DATE: 7/15/86 ELEV.:

Blow Counts*	Percent Passing No. 200 Sieve	MOISTURE CONTENT % OF DRY WT.	DRY DENSITY (PCF)	DEPTH (Ft.)	Description
				0	3" Asphaltic Concrete on 6" Aggregate Base
14					DARK BROWN GRAVELLY COARSE SAND (SW/SP); medium dense, slightly moist; FILL
50				5	ORANGE-BROWN GRAVELLY SAND (SP); dense, slightly moist, 2" maximum gravel.
					DARK BROWN SLIGHTLY SILTY GRAVELLY SAND (SP/SM); medium dense, slightly moist.
16				10	ORANGE-BROWN GRAVELLY COARSE SAND (SP/SW); medium dense, slightly moist.
					Water Level 7/15/86
					Very coarse grained, saturated.
14	8	12.8	111	15	DARK BROWN VERY FINE SAND (SP); medium dense, saturated.
32				20	DARK BROWN VERY GRAVELLY COARSE SAND (SP/GP); dense, saturated.
				25	
				30	
				40	

*BLOW COUNT: Number of blows required to drive a 2" inside diameter soil sampler 1' into undisturbed soil using a 140 lb. hammer falling 30".

J.H. KLEINFELDER & ASSOCIATES
GEOTECHNICAL CONSULTANTS • TESTING LABORATORIES

LOG OF BORING 1

PROPOSED SQUAW VALLEY INN EXPANSION
SQUAW VALLEY, CALIFORNIA

FILE NO: R-1665-1 DRAWN BY:

DATE: [illegible] CHKD. BY: [illegible]

PLATE 2

LOG OF Boring 2

EQUIPMENT: CME 55 Auger Drill Rig

DATE: 7/15/86 ELEV.:

Blow Counts	Percent Passing No. 200 Sieve	MOISTURE CONTENT % OF DRY WT.	DRY DENSITY (PCF)	DEPTH (Ft.)
28	3	13.0	98	5
49		6.9	106	10
59	5	9.2	114	15
31				20
				25
				30
				40

3" Asphalt Concrete on 5" Aggregate Base

DARK BROWN SLIGHTLY SILTY GRAVELLY SAND (SP); loose to medium dense, slightly moist; FILL.

TAN VERY COARSE GRAVELLY SAND (SP); medium dense slightly moist; FILL.

ORANGE-BROWN SANDY GRAVEL (GW); dense, dry to slightly moist, well graded; 3" maximum gravel diameter.

Water Level 7/15/86

GREY-BROWN MEDIUM TO COARSE SAND (SW).

GREY-BROWN VERY COURSE SAND (SW); dense, saturated.

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GEOTECHNICAL CONSULTANTS • TESTING LABORATORIES

FILE NO: R-1565-1 DRAWN BY:

DATE: 8-13-86 CHKD. BY: [illegible]

LOG OF BORING 2

PROPOSED SQUAW VALLEY INN EXPANSION
SQUAW VALLEY, CALIFORNIA

PLATE 3

LOG OF Boring 3

EQUIPMENT: CME 55 Auger Drill Rig

DATE: 7/15/86 ELEV.:

Blow Counts*	Percent Passing No. 200 Sieve	MOISTURE CONTENT % OF DRY WT.	DRY DENSITY (PCF)	DEPTH (Ft.)	SAMPLE LOCATION	DESCRIPTION
						DARK BROWN GRAVELLY SLIGHTLY SILTY SAND (SP); loose to medium dense, dry to slightly moist; FILL.
30						ORANGE-BROWN MEDIUM TO COARSE SAND (SP); medi dense to dense, slightly moist, some small gravel to 3/4"; FILL
				5		Becoming loose and more coarse below 4'.
7						Moist at 6-1/2'.
						ORANGE-BROWN VERY SANDY GRAVEL (GW/SW); dense moist.
				10		
43						
						ORANGE-BROWN GRAVELLY SAND (SP); dense, very moist to wet.
40						
				15		Water Level 7/15/86.
36						ORANGE-BROWN VERY SAND GRAVEL (GW/SW); dense, saturated.
						DARK BROWN FINE TO COARSE SAND (SW/SP); medium dense, saturated.
				20		
16						
						Very fine grained and loose below 24'.
				25		GRAY HIGHLY PLASTIC SILT (MH); medium, stiff, saturated.
5						RED-BROWN VERY COARSE GRAVELLY SAND (SP); dens saturated.
				30		
37						
				40		

J.H. KLEINFELDER & ASSOCIATES GEOTECHNICAL CONSULTANTS • TESTING LABORATORIES	LOG OF BORING 3	PLATE 4
FILE NO: R-1665-1 DRAWN BY: DATE: 8-13-86 CHKD. BY: 11	PROPOSED SQUAW VALLEY INN EXPANSION SQUAW VALLEY, CALIFORNIA	

LOG OF Boring 3

EQUIPMENT: CME 55 Auger Drill Rig

DATE: 7/15/86 ELEV.:

Blow Counts*	Percent Passing No. 200 Sieve	MOISTURE CONTENT % OF DRY WT.	DRY DENSITY (PCF)	DEPTH (Ft.)	SAMPLE LOCATION		
67							MEDIUM TO RED-BROWN SANDY GRAVEL (GP/SP); very dense, saturated.
				45			

J.H. KLEINFELDER & ASSOCIATES

GEOTECHNICAL CONSULTANTS • TESTING LABORATORIES

FILE NO: R-1665-1 DRAWN BY:

DATE: 8-3-86 CHKD. BY: [illegible]

LOG OF BORING 3

PROPOSED SQUAW VALLEY INN EXPANSION
SQUAW VALLEY, CALIFORNIA

PLATE 5

LOG OF Boring 4

EQUIPMENT: CME 55 Auger Drill Rig

DATE: 7/15/86 ELEV.:

Blow Counts*	Percent Passing No. 200 Sieve	MOISTURE CONTENT % OF DRY WT.	DRY DENSITY (PCF)	DEPTH (Ft.)	Description
14	13				DARK BROWN GRAVELLY SILTY SAND (SM); medium dense, slightly moist; FILL. ORANGE-BROWN COARSE SAND (SW); medium dense, dry to slightly moist; FILL.
				5	BROWN VERY GRAVELLY SAND (SP/GP); very dense moist.
67		5.3	114		
				10	DARK BROWN VERY CLAYEY SAND (SC/CL); loose, very moist to wet.
4 8	43	38.8	82		ORANGE BROWN GRAVELLY SAND (SP); very moist to wet, 2" maximum gravel size.
				15	
27					
				20	No Free Water Encountered
				25	
				30	
				40	

J.H. KLEINFELDER & ASSOCIATES

GEOTECHNICAL CONSULTANTS • TESTING LABORATORIES

FILE NO: R-1665-1 DRAWN BY: ______

DATE: 8-13-86 CHKD. BY: [illegible]

LOG OF BORING 4

PROPOSED SQUAW VALLEY INN EXPANSION

SQUAW VALLEY, CALIFORNIA

PLATE

6

LOG OF Boring 5

EQUIPMENT: CME 55 Auger Drill Rig

DATE: 7/15/86 ELEV.:

Blow Counts*	Percent Passing No. 200 Sieve	MOISTURE CONTENT % OF DRY WT.	DRY DENSITY (PCF)	DEPTH (Ft.)
9		7.4	96	
				5
26				
				10
11	3	26.5	91	
				15
3	46	34.2	87	
				20
22				
				25
15		41.1	79	
				30
41				
44				
95				
				40

SAMPLE LOCATION

DARK BROWN SILTY GRAVELLY SAND (SP/SM); loose, slightly moist, 2-1/2" maximum gravel size; FILL.

ORANGE-BROWN GRAVELLY COARSE SAND (SP); medium dense, slightly moist, 3" maximum gravel size.

BROWN FINE SAND (SW); loose to medium dense, moist, color change at 11' to orange brown, and coarse grained sand.

DARK BROWN VERY FINE VERY SILTY SAND (SM/ML); loose, wet to saturated.

Water Level 7/16/86.

BROWN SANDY, GRAVEL (GP); medium dense to dense, saturated.

BROWN FINE SAND (SP); medium dense, saturated.

ORANGE BROWN VERY SANDY GRAVEL (SP/SP); medium dense, saturated.

MEDIUM BROWN VERY FINE SLIGHTLY PLASTIC CLAYEY SAND (SC/CL); loose, saturated.

ORANGE-BROWN CLAYEY GRAVELLY SAND (SP/SC); medium dense to dense, saturated, clay decreased in depth.

BROWN TO ORANGE BROWN GRAVELLY SAND (SP); dense to very dense, saturated; with very fine sand interbeds.

J.H. KLEINFELDER & ASSOCIATES GEOTECHNICAL CONSULTANTS • TESTING LABORATORIES	LOG OF BORING 5	PLATE 7
FILE NO: R-1665-1 DRAWN BY: DATE: 8-3-86 CHKD. BY: RJ	PROPOSED SQUAW VALLEY INN EXPANSION SQUAW VALLEY, CALIFORNIA	

LOG OF Boring 6

EQUIPMENT: CME 55 Auger Drill Rig

DATE: 7/15/86 ELEV.:

Blow Counts*	Percent Passing No. 200 Sieve	Moisture Content % of Dry Wt.	Dry Density (PCF)	Depth (Ft.)	Sample Location	Description
						BROWN SILTY VERY SANDY GRAVEL (GW/SW); loose in upper 2', medium dense to very dense, slightly moist to moist below 2', 2-1/2" maximum gravel size.
49						
				5		
						ORANGE-BROWN GRAVELLY SAND (SP); medium dense to very dense, very moist to saturated.
				10		
38						
						Water Level 7/16/86 (Approximate)
						Water Level 7/18/86 (Stabilized)
				15		
32						
				20		
51						
				25		
				30		
				40		

J.H. KLEINFELDER & ASSOCIATES
GEOTECHNICAL CONSULTANTS • TESTING LABORATORIES

FILE NO: R-1565-1 DRAWN BY:

DATE: 8-13-86 CHKD. BY: CJ

LOG OF BORING 6

PROPOSED SQUAW VALLEY INN EXPANSION
SQUAW VALLEY, CALIFORNIA

PLATE 8

Blow Count*	Percent Passing No. 200 Sieve	MOISTURE CONTENT % OF DRY WT.	DRY DENSITY (PCF)	DEPTH (Ft.)	SAMPLE LOCATION
20	9				
				5	
				10	
				15	

LOG OF Boring 7

EQUIPMENT: CME 55 Auger Drill Rig

DATE: 7/15/86 ELEV.:

2-1/2" AC (No aggregate base).

BROWN SILTY FINE TO COARSE SAND (SP/SM); medium dense, slightly moist; FILL.

LOG OF

EQUIPMENT:

DATE: ELEV.:

				DEPTH (Ft.)	
				5	
				10	
				15	

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LOG OF BORING 7

PLATE 9

FILE NO: R-1665-1 DRAWN BY:

DATE: 8-13-86 CHKD. BY: [illegible]

PROPOSED SQUAW VALLEY INN EXPANSION
SQUAW VALLEY, CALIFORNIA

APPENDIX C

Suggested Specifications
For
Earthwork and Pavement Construction

APPENDIX C

SUGGESTED SPECIFICATIONS FOR EARTHWORK AND PAVEMENT CONSTRUCTION PROPOSED PLUMPJACK SQUAW VALLEY INN ADDITION PLACER COUNTY, CALIFORNIA

1.0 GENERAL

1.1 Scope - The work done under these specifications shall include clearing, stripping, removal of unsuitable material, excavation, installation of subsurface drainage, preparation of natural soils, placement and compaction of on-site and imported fill material, and placement and compaction of pavement materials.

1.2 Contractor's Responsibility - A geotechnical investigation was performed for the project by Kleinfelder dated December 27, 2000. The Contractor shall attentively examine the site in such a manner that he can confirm existing surface conditions with those presented in the soils report. He shall satisfy himself that the quality and quantity of exposed materials and subsurface soil or rock deposits have been satisfactorily represented by the Soils Geotechnical Engineer's report and Civil Engineer's drawings. Any discrepancy that may be of prior knowledge to the Contractor or that is revealed through his investigations shall be made available to the owner. It is the Contractor's responsibility to review the attached report prior to construction. The selection of equipment for use on the project and the order of work will similarly be his responsibility such that the requirements included in following sections have been met.

1.3 Geotechnical Engineer - The work covered by these specifications shall be observed and tested by the Geotechnical Engineer, Kleinfelder, who shall be hired by the Owner. The Geotechnical Engineer will be present during the site preparation and grading to observe the work and to perform the tests necessary to evaluate material quality and compaction. The Geotechnical Engineer shall submit a report to the Owner, including a tabulation of all tests performed. The costs of retesting of unsuitable work performed by the Contractor shall be deducted from the payments to the Contractor.

1.4 Standard Specifications - Where referred to in these specifications, "Standard Specifications" shall mean the current State of California Department of Transportation (Caltrans) Standard Specifications.

1.5 ASTM Specifications - Where referenced to in these specifications, ASTM test methods shall refer to the American Society for Testing and Materials Standards, current annual edition.

1.6 Compaction Test Method - Where referred to herein, relative compaction outside of the state right-of-ways shall mean the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by ASTM D1557 Compaction Test Procedure. Optimum moisture content shall mean the moisture content of maximum dry density as determined above.

2.0 SITE PREPARATION

2.1 Clearing - Areas to be graded shall be cleared and grubbed of all vegetation and debris. These materials shall be removed from the site by the Contractor.

2.2 Stripping - Surface soils containing roots and organic matter shall be stripped from areas to be graded and stockpiled or discarded as directed by the Owner. In general, the depth of stripping of the topsoil will be approximately four inches. Deeper stripping, where required to remove weak soils or accumulations of organic matter, shall be performed when determined by the Geotechnical Engineer. Strippings shall be removed from the site or stockpiled at a location designated by the Owner.

2.3 Removal of Debris - Existing pavements, trash, and debris in the areas to be graded shall be removed prior to the placing of any compacted fill. Portions of any existing fills that are suitable for use in compacted fill may be stockpiled for future use. All organic material, topsoil, expansive soils, oversize material or other unsuitable material shall be removed from the site by the Contractor or disposed of at a location on site, if so designated by the Owner.

2.4 Ground Surface - The ground surface exposed by stripping shall be scarified to a depth of 6 inches, moisture conditioned to the proper moisture content for compaction, and compacted to a minimum of 90% relative compaction. Recompaction shall be approved by the Geotechnical Engineer prior to placing fill.

3.0 EXCAVATION

3.1 General - Excavations shall be performed to the lines and grades indicated on the plans.

The data presented in the soils report is for information only and the Contractor shall make his own interpretation with regard to the methods and equipment necessary to perform the excavation and to obtain material suitable for fill.

3.2 Materials - Soils which are removed and are unsuitable for fill should be placed in non-structural areas of the project. Where necessary, these soils may be placed in deeper fills if approved by the Geotechnical Engineer.

All oversized rocks and boulders that cannot be incorporated in the work by placing in embankments or used as riprap or for other purposes shall be removed from the site by the Contractor.

3.3 Treatment of Exposed Surface - The ground surface exposed by excavation shall be scarified to a depth of 6 inches, moisture conditioned to the proper moisture content for compaction, and compacted as required for compacted fill. Recompaction shall be approved by the Geotechnical Engineer prior to placing fill.

3.4 Rock Excavation - Where solid rock is encountered in excavation, it shall be loosened and broken up so that no solid ribs, projections, or large fragments will be within 6 inches of the surface of the final subgrade.

4.0 COMPACTED FILL

4.1 Materials - Fill material shall consist of suitable on site or imported fill. All materials used for structural fill shall be reasonably free of organic material, have a liquid limit less than 25,. a plasticity index less than 12, 100% passing the 6-inch sieve, and less than 25 passing the No. 200 sieve.

4.2 Placement - All fill materials shall be placed in layers of 8 inches or less in loose thickness and uniformly moisture conditioned. The lift should then be compacted with a sheepsfoot roller or other approved compaction equipment to achieve at least 90% relative compaction in areas under structures, and to at least 85% in undeveloped areas. A relative compaction of at least 90% should be achieved in utility trench backfill and under pavements on private property. No fill material shall be placed, spread or rolled while it is frozen or thawing, or during unfavorable weather conditions.

4.3 Benching - Fill placed on slopes steeper than 5 horizontal to 1 vertical shall be keyed into firm, native soils or rock by a series of benches. Benching can be conducted simultaneously with placement of fill. However, the method and extent of benching shall be checked by the Geotechnical Engineer.

4.4 Compaction Equipment - The Contractor shall provide and use sufficient equipment of a type and weight suitable for the conditions encountered in the field. The equipment shall be capable of obtaining the required compaction in all areas, including those that are inaccessible to ordinary rolling equipment.

4.5 Recompaction - When, in the judgment of the Geotechnical Engineer, sufficient compaction effort has not been used, or where the field density tests indicate that the required compaction or moisture content has not been obtained, or if "pumping" or other indications of instability are noted, the fill shall be reworked and recompacted as needed to obtain a stable fill at the required density and moisture content prior to placing additional fill materials.

4.6 Responsibility - The Contractor shall be responsible for maintenance and protection of all embankments and fills made during the contract period and shall bear the expense of replacing any portion which has become displaced due to carelessness, negligent work, or failure to take proper precautions.

5.0 UTILITY TRENCH BEDDING AND BACKFILL

5.1 Material - Pipe bedding shall be defined as all material within 6 inches of the perimeter of the pipe. Material for use as bedding shall consist of clean, granular material free of clay and organic material and be such a size that 90 to 100% will pass a No. 4 sieve and not more than 5% will pass a No. 200 sieve.

Backfill should be classified as all material within the remainder of the trench. Backfill shall conform to the following gradation:

Percent Passing	Sieve Size
100	3-Inch
35 - 100	No. 4
20 - 100	No. 30

5.2 Placement and Compaction - Pipe bedding shall be placed in thin layers not exceeding 8 inches in loose thickness, conditioned to the proper moisture content for compaction, and compacted to at least 90% relative compaction. All other trench backfill shall be placed in thin layers not exceeding 8 inches in loose thickness, conditioned to the proper moisture content, and compacted as required for adjacent fill. If not specified, backfill should be compacted to at least 90% relative compaction in areas under structures, utilities, roadways, parking areas and concrete flatwork, and to 85% relative compaction in undeveloped areas.

6.0 SUBSURFACE DRAINAGE

6.1 General - Subsurface drainage should be constructed as shown on the plans. Drainage pipe should meet the requirements set forth in the Standard Specifications.

6.2 Materials - Permeable "drain rock" material used for subdrainage should meet the following gradation requirements or consist of other material approved by the Geotechnical Engineer.

Sieve Size	Percentage Passing
2″	100
1-1/2″	95 - 100
3/4″	50 - 100
3/8″	15 - 55
No. 4	0 - 25
No. 100	0 - 5
No. 200	0 - 3

6.3 Geotextile Fabric - Nonwoven filter fabric should be placed between the permeable drain rock and native soils. Filter cloth with an equivalent opening size greater than the No. 100 sieve size and a grab strength not less than 100 pounds should be used. We should be consulted on a specific basis when a particular fabric is chosen so that compliance to the above recommendations can be verified.

6.4 Placement & Compaction - Drain rock shall be placed in thin layers not exceeding 8 inches in loose thickness and compacted as required for adjacent fill, but in no case will be less than 85% relative compaction. Placement of geotextile fabric will be in accordance with the manufacturer's specifications, and should be checked by the Geotechnical Engineer.

7.0 AGGREGATE BASE FOR CONCRETE SLABS

7.1 Material - Aggregate base for concrete slabs shall consist of clean free-draining sand, gravel or crushed rock conforming to the following gradation:

Sieve Size	Percent Passing
1"	100
3/8"	30 - 100
No. 200	0 - 10

7.2 Placement - Aggregate base shall be compacted and kept moist until placement of concrete. Compaction shall be by suitable vibrating compactors. Aggregate base shall be placed in layers not exceeding 8 inches in thickness. Each layer shall be compacted by at least four passes of the compaction equipment or until 95% relative compaction has been obtained.

8.0 SUBGRADE AND AGGREGATE BASE FOR PAVED AREAS

8.1 Subgrade Preparation - After completion of the utility trench backfill and prior to placement of aggregate base, the upper 6 inches of subgrade soil shall be uniformly compacted to at least 95% relative compaction. This may require scarifying, moisture conditioning, and compacting in both cut and fill areas.

8.2 Aggregate Base - Aggregate materials shall meet the requirements of the appropriate sections of the "Standard Specifications" for Class 2 aggregate base. The aggregate base materials must be approved by the Geotechnical Engineer prior to use.

After the subgrade is properly prepared, the aggregate base shall be placed in layers, moisture conditioned as necessary, and compacted by rolling to at least 95% relative compaction. The compacted thickness of aggregate base shall be shown on the approved plans.

9.0 ASPHALT CONCRETE PAVEMENT

9.1 Thickness - The compacted thickness of asphalt concrete shall be as shown on the approved plans.

9.2 **Materials** - Aggregate materials for asphalt concrete shall conform to the requirements listed for Type A or Type B bituminous aggregates in Section 39 of the "Standard Specifications" and utilize AR-4000 grade of asphalt concrete. The Contractor shall submit a proposed asphalt concrete mix design to the Owner for review and approval prior to paving. The mix design shall be based on the Hveem Method.

Where prime coat is specified, the type and grade of asphalt for use as prime coat shall be MC70 or MC250 with an application rate of 0.20 to 0.30 gallons per square yard. The type and grade of asphalt for use as tack coat shall be SS-1 or SS1-h with an application rate of 0.10 to 0.15 gallons per square yard.

The type and grade of asphalt for use as seal coat shall be CSS-1h or CSS-1 with an application rate of 0.15 to 0.20 gallons per square yard. Sand blotter, if needed to prevent "pick-up", shall be spread at a rate of 10 to 15 pounds per square yard.

9.3 **Placement** - The asphalt concrete material and placement procedures shall conform to appropriate sections of the "Standard Specifications".

APPENDIX D

Application for Authorization to Use

APPENDIX D
APPLICATION FOR AUTHORIZATION TO USE

PRELIMINARY GEOTECHNICAL REPORT
PROPOSED PLUMPJACK SQUAW VALLEY INN ADDITION
PLACER COUNTY, NEVADA

Kleinfelder, Inc.
4875 Longley Lane, Suite 100
Reno, Nevada 89502

To whom it may concern:

Applicant understands and agrees that the "Preliminary Geotechnical Investigation Report, Proposed PlumpJack Squaw Valley Inn Addition, Placer County, Nevada," dated December 27, 2000, Job No. 30-2583-02.001, for the subject site is a copyrighted document, that Kleinfelder, Inc. is the copyright owner and that unauthorized use or copying of said document for the subject site is strictly prohibited without the express written permission of Kleinfelder, Inc. Applicant understands that Kleinfelder, Inc. may withhold such permission at its sole discretion, or grant permission upon such terms and conditions as it deems acceptable.

Applicant agrees to accept the contractual terms and conditions between Kleinfelder, Inc. and K. B. Foster Civil Engineering, Inc. originally negotiated for preparation of this document. Use of this document without permission releases Kleinfelder, Inc. from any liability that may arise from use of this report.

To be Completed by Applicant

____
(company name)

____
(address)

____
(city, state, zip)

____________ ____________
(telephone) *(FAX)*

By: ________________________

Title: ________________________

Date: ________________________

For Kleinfelder, Inc.'s use only

________ approved for re-use with additional fee of $ ____________
________ disapproved, report needs to be updated

By: ________________________
(Kleinfelder, Inc. project manager)

Date: __________________

Sc AW VALLEY INN
96198

Client: CNCML

GOETECHNICAL INVESTIGATION FOR
PROPOSED SQUAW VALLEY INN PROJECT
SQUAW VALLEY, CALIFORNIA

Date: August 14, 1986

J. H. KLEINFELDER & ASSOCIATES
GEOTECHNICAL CONSULTANTS • MATERIALS TESTING
LAND AND WATER RESOURCES

J. H. KLEINFELDER & ASSOCIATES
GEOTECHNICAL & ENVIRONMENTAL CONSULTANTS • MATERIALS TESTING
LAND & WATER RESOURCES
3189 MILL STREET • RENO, NV 89502
(702) 323-7182

August 14, 1986
File: R-1665-1

CNCML
22 Battery Street, Suite 202
San Francisco, CA 94111

Attention: Mr. Larry Chazen

Subject: Geotechnical Investigation for the Proposed
Squaw Valley Inn Project
Squaw Valley, California

Gentlemen:

The attached report presents the results of our geotechnical investigation for the proposed Squaw Valley Inn project to be located adjacent to the existing Squaw Valley Inn, Squaw Valley, California.

If you have any questions about our report or need additional information, please contact us. We appreciate this opportunity to be of service to you, and look forward to future endeavors.

Yours very truly,

J. H. KLEINFELDER & ASSOCIATES

Christine M. Welch, C.E.
Project Engineer

Chris D. Spandau, C.E.
Project Engineer

CMW:CDS:tc

cc: Gene T. Takigawa, AIA (3 bound, 1 unbound)

TABLE OF CONTENTS

GEOTECHNICAL INVESTIGATION FOR PROPOSED SQUAW VALLEY INN PROJECT SQUAW VALLEY, CALIFORNIA

I. INTRODUCTION

A. Project Description

This report presents the results of our geotechnical investigation for the proposed expansion of the Squaw Valley Inn in Squaw Valley, California. We understand that construction will occur in two phases. Phase 1 will include construction of a 2-story, wood frame, multi-purpose building with a concrete slab-on-grade floor. Phase 1 will also include a new asphaltic concrete paving for 79 parking spaces, as well as service corridors and entrance/exit drives off of Squaw Peak and Squaw Valley Roads.

Conventional shallow footings are anticipated for the Phase 1 structure. At this time, structural framing design has not been completed. Discussion with Mr. Garry Knott, structural engineer with Carter Engineering, indicate that maximum wall loads may be as great as 10 kips per linear foot, if isolated column foundations are not used. If column foundations are used to support roof loads, wall loads should be reduced to approximately 3 to 4 kips per linear foot, and maximum column loads of 80 kips can be anticipated. Cuts and fills are anticipated to vary from 0 to 2 feet during site grading.

Phase 2 construction is conceptual at this time, and may include construction of a 3 to 4 story hotel structure with one underground parking level. Since details regarding building location, type of construction etc. are still in the planning phases, our investigation for Phase 2 construction is intended to provide conceptual design information only. Additional analysis and possibly field exploration will be necessary once the scope of the Phase 2 development is more clearly defined.

B. Purpose and Scope of Work

The purpose of this investigation is to evaluate the feasibility of the proposed development with respect to subsurface conditions, and to provide our recommendations and opinions concerning the following:

Phase I Construction

- Soils criteria for foundation design, including allowable bearing pressures, passive soil resistance to lateral loads, and anticipated settlements;
- Anticipated conditions for site preparation and earthwork construction including preparation of suggested specifications for earthwork and paving operations;
- Subgrade and aggregate base requirements for concrete slab-on-grade floors;
- Preliminary asphaltic concrete pavement sections.

Phase 2 Construction

- Feasibility of the proposed construction from a geotechnical standpoint;
- Recommended foundation types and anticipated ranges of foundation design parameters;
- Anticipated earthwork construction conditions.

The scope of work consisted of field exploration, laboratory soils testing, engineering analyses, and preparation of this report.

C. Authorization

Authorization to proceed with our work on this project was given by Mr. Chazen on July 10, 1986 in the form of a signed standard form of agreement.

II. METHODS OF INVESTIGATION

A. Field Exploration

The subsurface exploration consisted of drilling 2 borings within the proposed multi-purpose building, 4 borings in the Phase 2 area and a shallow boring within the northwestern part of the site where a new paved area is planned. Borings were advanced to depths up to 41-1/2 feet below the existing ground surface, using hollow stem auger drilling methods. Locations of the borings, as shown on the Site Plan, Plate 1, were approximately determined by pacing from existing structures and from property lines. These locations should be considered accurate only to the degree implied by the method used. A field engineer logged the soil conditions encountered and obtained both bulk and relatively undisturbed drive samples for laboratory testing.

Soil conditions encountered are presented on the Boring Logs, Plates 2 through 9. A description of the Unified Soil Classification System used to identify the site soils and a Key to the Logs are presented on Plate 10.

B. Laboratory Testing

Our laboratory program consisted of tests for soil classification, unit weight and moisture content, consolidation and strength characteristics, and resistance (R) value tests for use in flexible pavement design. Laboratory test results can be found on the boring logs and on Plates 10 through 17.

III. DISCUSSION, CONCLUSIONS AND RECOMMENDATIONS

A. Discussion

1. Site Description

The property is bordered on the east by Squaw Valley Road, on the south by Squaw Peak Road, and by Squaw Creek along the northern edge. The eastern half of the site is currently occupied by the two-story Squaw Valley Inn Hotel structure with an adjoining pool area to the north of the hotel. The areas to the east and north of the hotel are paved with asphalt concrete. It is known that fillsoils were placed on the site to allow for parking or other non-structural use during the 1960 Olympics.

The site is generally flat with drainage towards the northern edge of the property. The overall slope of the site is less than 2 percent. Within the western third of the Phase II site is an intermittent stream channel which contains loose gravels. The Phase II site is currently used as a overflow gravel parking area and contains numerous trees.

2. Subsurface Conditions

Near surface soils encountered in our borings consist of a relatively loose, slightly moist, mixture of clean sand and gravel clean which extends to depths of 5 to 9 feet below the existing ground surface. This unit is believed to be fill material which was placed prior to the 1960 Winter Olympics at Squaw Valley. The original ground surface appears to have been stripped of vegetation prior to fill placement, since we did not observe any topsoil or vegetation in our borings. The fill soils did not appear to contain deleterious amounts of organic material.

Underlying the loose fill soils, we generally encountered medium dense to very dense clean sands and gravels in the vicinity of the proposed multi-purpose facility. Native soils became more silty and clayey in our borings located to the west of the Phase 1 structure, and contained layers of silty and clayey sands, interbedded with the previously described clean granular soils. A highly plastic silt unit, encountered between 25-1/2 and 27-1/2 feet in Boring 3, was the only fine grained soil logged in our borings. This unit does not appear to be continuous across the site.

At a few locations and depths, (B-4 @ 10-1/2 feet and B-5 @ 15 feet) unusually loose soils were encountered. These loose areas generally occurred within units containing a significant proportion of fine grained material.

At the time of our field exploration, water was encountered at depths of approximately 14 to 16 feet below the ground surface, except in Boring 4, where no free water was encountered immediately after drilling to 20-1/2 feet. It is probable that water would have eventually flowed into this bore hole, had it been left open for several hours following drilling. A piezometer was installed in Boring 6, to allow for future monitoring of groundwater levels. Based on rusty color mottling of site soils below approximately 11 to 13 feet, we feel that the periodic high groundwater level may rise to within 10 to 12 feet of the ground surface.

B. Conclusions

Based on the results of our preliminary investigation, we feel that Phase 1 construction can occur as planned, from a geotechnical engineering standpoint. We feel that existing, relatively loose fill soils should be partially or completely removed beneath foundations and slabs and replaced as properly compacted structural fill, as described in subsequent portions of this report.

We believe that the major concerns to Phase 2 development from a soils engineering standpoint are the possible presence of groundwater within excavations for subterranean parking, and the presence of locally loose or silty sand pockets. These "pockets" could undergo significant consolidation and result in differential settlements of shallow, conventional spread foundations. If loads for the Phase 2 hotel structure are large and differential settlement tolerances are small, the use of a deep pile foundation should be considered. Our past work on the Dempsey project just south of the Phase 2 area encountered similiar conditions. Close work between the soils structural engineers allowed this three story building with subterranean parking to be constructed on shallow spread footings.

C. Recommendations

1. Phase 1 Construction

A. Site Preparation and Earthwork

Site preparation and grading should conform to the requirements contained in this report and in the suggested specifications, Appendix A of this report. We anticipate that site grading can be performed with conventional earthmoving equipment. Prior to construction, the existing asphalt concrete pavements and aggregate base material should be stripped and removed from construction areas. Outside of foundation or building slab areas, prior to fill placement, the exposed old fill soils should be scarified to a minimum depth of 6 inches, moisture conditioned as necessary, and densified to a minimum of 90 percent relative compaction in accordance with the ASTM D1557 Compaction Test Method.

Where fill is necessary, materials should meet the requirements listed for "structural fill" in Appendix A. Existing site soils should generally meet these qualifications for structural fill. Fill placement and compaction requirements are presented in Appendix A should be followed.

Within the multi-purpose building footprint, existing loose fill soils should be over excavated, moisture conditioned to near optimum moisture content, replaced in thin lifts and densified to a minimum of 90 percent relative compaction, as described in subsequent sections of this report.

B. Foundations

We recommend that the proposed structure be supported on conventional, continuous wall or isolated spread foundations. Exterior foundations should be bottomed a minimum of 30 inches below lowest adjacent grade for frost protection and confinement. Interior footings should be bottomed at least 12 inches below grade for confinement. Foundation dimensions should conform to requirements in the Uniform Building Code.

It is anticipated that relatively loose fill soils will be encountered at planned footing elevations. In order to reduce the potential for differential settlement of foundations, we recommend removing a minimum of 2 feet of these soils beneath foundations. Overexcavations should extend a minimum of 2 feet beyond all foundation lines. The exposed soils should be proof rolled to expose any loose or yielding soils, and any such material should be removed or compacted to at least 90 percent relative compaction. The overexcavations should be replaced with properly moisture conditioned and compacted native or imported soils meeting requirements for structural fill.

Foundations bottomed on compacted fill soil as described above can be designed using a maximum soil bearing pressure of 2500 pounds per square foot for dead plus long-term live loads. The allowable bearing pressure may be increased by one-third for total loading conditions, including wind and seismic forces.

Alternatively, if the depth of overexcavation and replacement is increased to 4 feet, a bearing pressure of 3500 psf may be used.

Existing native and fill soils are relatively dry and cohesionless and are therefore likely to cave and slough into open excavations, unless adequate shoring or side slopes are maintained.

Reinforcement steel requirements for foundations should be designed by the structural engineer.

Passive resistance to lateral earth pressures may be calculated using an equivalent fluid pressure of 350 pounds per cubic foot or a coefficient of friction of 0.35 applied to vertical dead loads.

We estimate that total post-construction settlement of footings designed and constructed in accordance with our recommendations will be on the order of 3/4 inch, with approximate differential settlement of 1/2 inch or less.

C. Concrete Slab-on-Grade

We recommend that concrete slabs should be supported on a 12 inch thick fill mat consisting of suitable on-site or imported fill which has been compacted to a minimum of 90 percent relative compaction. Slabs should be directly underlain by at least 4 inches of base rock consisting of clean free-draining sand, gravel, or crushed rock to provide a capillary break to migrating soil moisture. In addition, we recommend that a impervious membrane be placed above the base rock. The membrane should consist of 6 mil visqueen or equivalent and the membrane be protected with at least 2 inches of clean sand over the membrane. This sand be kept moist until placement of concrete to facilitate curing.

D. Pavement Sections

Recommended pavement structural sections for the project are presented in Table 1. Structural sections for several traffic loadings are presented. A Traffic Index of 4.5 generally corresponds to light traffic loadings and frequencies, such as for automobile parking areas. A Traffic Index of 6 is generally used for moderate traffic loadings and frequencies, such as for main entrance and exit ways and delivery truck corridors.

TABLE 1

Traffic Index	Recommended Structural Section
4.5	2.5" of asphalt concrete on 4 " of aggregate base
6	3.5" of asphalt concrete on 4 " of aggregate base

Placement and compaction procedures for materials and construction should conform to the suggested specifications contained in Appendix A of this report. The sections presented in Table 1 are based on an R-value test performed on a selected sample obtained during our investigation and should be considered preliminary in nature. We recommend verification of soil conditions as construction progresses so that appropriate revisions can be made, if necessary.

2. Phase 2 Construction

A. General

We feel that the construction of a 3 to 4 story hotel structure with underground parking at the proposed Phase 2 site is feasible from a geotechnical standpoint. Building location loads and types are unknown at this time; therefore, subsequent recommendations and conclusions are presented for preliminary planning purposes only and are not intended for use in design. An additional geotechnical investigation will be required once final plans have been developed.

There are indications that the seasonal high groundwater level at the site may be on the order of 10 to 12 feet below the ground surface. It may be prudent to design any underground structure such that footings do not extend deeper than 8 to 10 feet below the existing ground surface, if possible. This would reduce construction costs, delays and difficulties by allowing work to be performed in the dry soil horizons, and would reduce the potential for post-construction drainage and uplift problems. Caving of excavations both below and above the water table is possible.

B. Foundations

We believe that conventional shallow spread foundations will be appropriate for light to moderate loading conditions, where isolated column foundations will not exceed 200 kips and continuous wall foundations will not exceed 10 kips per linear foot. It is probable that some foundations will be required to be over-excavated and underlain by several feet of structural fill, in order to reduce range of possible differential settlements. No foundations should be bottomed directly on the loose fill soils now present within the upper 5 to 9 feet at the Phase 2 portion of the site.

Allowable soil bearing pressures on the order of 2000 to 3500 psf can be anticipated for dead plus long-term loads, with total post-construction settlement of foundations being limited to 3/4 inch, and differential settlement limited to 1/2 inch.

If very heavy loads will be developed in excess of those described above, a deep foundation system, such as driven piles or cast-in-place piers, should be investigated.

C. Anticipated Earthwork Construction Conditions

We feel that the proposed construction may be significantly impeded by the possible presence of groundwater, if excavations for underground improvement extend below approximately 10 feet below the existing ground surface. This is particularly true if construction is scheduled for the spring or early summer season, when groundwater levels are at their shallowest depth. Saturated, unstable soil may be encountered in deep foundation or utility trenches and dewatering may be required.

Localized pockets of soft or loose soils may be anticipated throughout the project site. These materials, where encountered, will require overexcavation and replacement with structural fill. Since the site soils above aproximately 10 feet are relatively dry and cohesionless, excavations will need to be properly shored or sloped back to reduce caving and/or sloughing.

IV. LIMITATIONS AND ADDITIONAL SERVICES

A. Limitations

Recommendations contained in this report are based on the field explorations, laboratory tests, and our understanding of the proposed construction. The soils data used in the preparation of this report were obtained from borings made for this investigation. It is possible that variations in the soils exist between points explored. The nature and extent of soil variations may not be evident until construction occurs. If any soil conditions are encountered at this site which are different than those described in this report, our firm should be immediately notified so that we may make any necessary revisions to recommendations contained in this report. In addition, if the scope of the proposed loading or locations of structures changes from that described in this report, our firm should be notified.

This report has been prepared for design purposes, for specific application to the Squaw Valley Inn multi-purpose building project and for preliminary planning purposes for all Phase 2 construction. This report has been prepared in accordance with the generally accepted standard of practice at the time the report was written. No other warranty, expressed or implied, is made.

It is the CLIENT'S responsibility to see that all parties to the project including the designer, contractor, subcontractors, etc. are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the Contractor's option and risk.

B. Additional Services

The recommendations made in this report are based on the assumption that an adequate program of tests and observations will be made during the construction to verify compliance with these recommendations. These tests and observations should include, but not necessarily be limited to, the following:

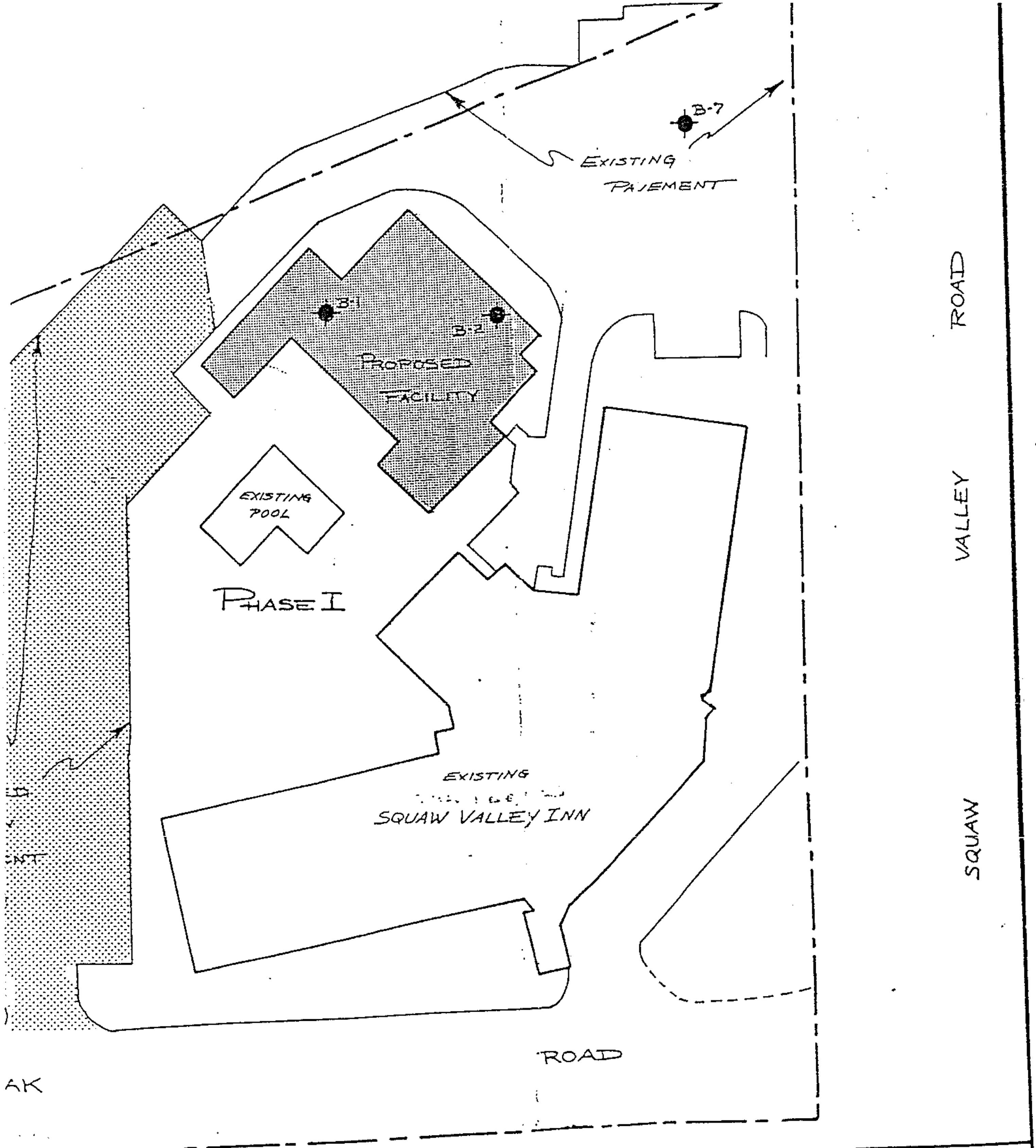
1. Observations and testing during site preparation and earthwork.

2. Observation of footing trench excavations.

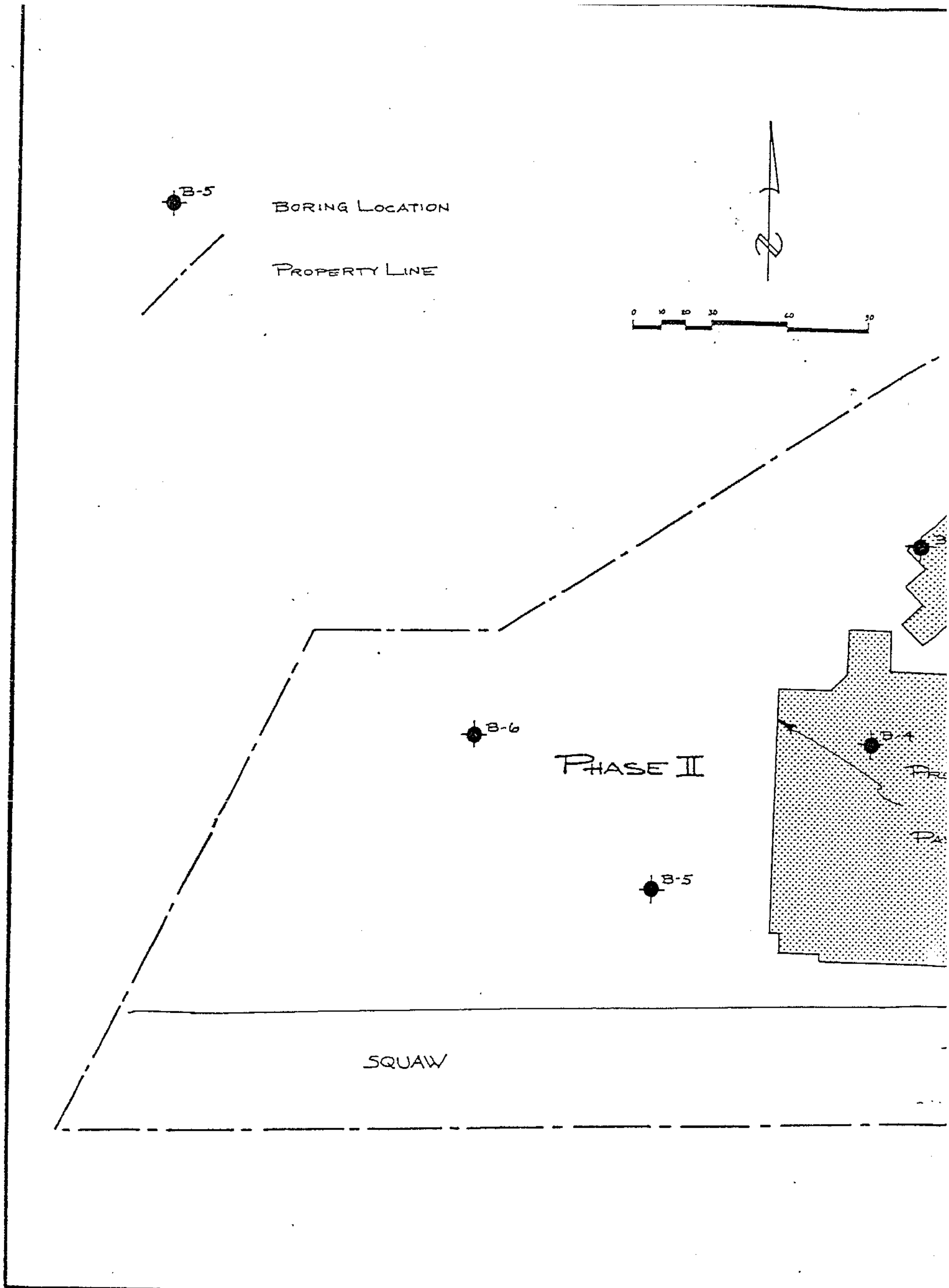
3. Consultation as may be required during construction.

We also recommend that project plans and specifications be reviewed by us to verify compatibility with our conclusions and recommendations. Additional information concerning the scope and cost of these services can be obtained from our office.

The review of plans and specifications and the field observation and testing by J. H. Kleinfelder & Associates are an integral part of the conclusions and recommendations made in this report. If we are not retained for these services, the client agrees to assume J. H. Kleinfelder & Associates' responsibility for any potential claims that may arise during construction.



J. H. KLEINFELDER & ASSOCIATES GEOTECHNICAL CONSULTANTS — ENGINEERING LABORATORIES	PROPOSED SQUAW VALLEY INN EXPANSION SQUAW VALLEY, CALIFORNIA
DRAWN BY: LHS DATE: 8-5-86 CHECKED BY: DATE:	PROJECT NO. R-1665-1 PLATE NO. 1



B-5
BORING LOCATION
PROPERTY LINE
0 10 20 30 40 50
B-6
PHASE II
B-5
B-4
SQUAW

LOG OF Boring 1

EQUIPMENT: CME 55 Auger Drill Rig

DATE: 7/15/86 **ELEV.:**

Blow Counts*	Percent Passing, No. 200 Sieve	MOISTURE CONTENT % OF DRY WT.	DRY DENSITY (PCF)	DEPTH (Ft.)	SAMPLE LOCATION	
						3" Asphaltic Concrete on 6" Aggregate Base
14						DARK BROWN GRAVELLY COARSE SAND (SW/SP); medium dense, slightly moist; FILL
				5		
50						ORANGE-BROWN GRAVELLY SAND (SP); dense, slightly moist, 2" maximum gravel.
						DARK BROWN SLIGHTLY SILTY GRAVELLY SAND (SP/SM); medium dense, slightly moist.
16				10		ORANGE-BROWN GRAVELLY COARSE SAND (SP/SW); medium dense, slightly moist.
						Water Level 7/15/86
				15		Very coarse grained, saturated.
14	3	12.8	111			DARK BROWN VERY FINE SAND (SP); medium dense, saturated.
				20		DARK BROWN VERY GRAVELLY COARSE SAND (SP/GP); dense, saturated.
32						
				25		
				30		
				40		

*BLOW COUNT: Number of blows required to drive a 2" inside diameter soil sampler 1' into undisturbed soil using a 140 lb. hammer falling 30".

J.H. KLEINFELDER & ASSOCIATES
GEOTECHNICAL CONSULTANTS • TESTING LABORATORIES

FILE NO: R-1665-1 DRAWN BY:

DATE: [illegible] CHKD. BY: [illegible]

LOG OF BORING 1

PROPOSED SQUAW VALLEY INN EXPANSION
SQUAW VALLEY, CALIFORNIA

PLATE 2

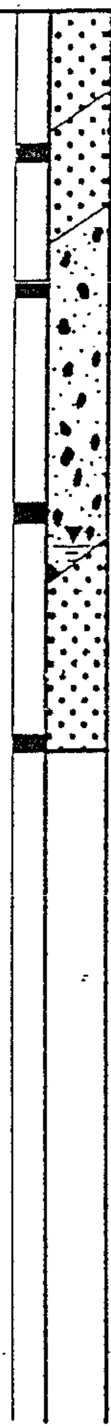
LOG OF Boring 2

EQUIPMENT: CME 55 Auger Drill Rig

DATE: 7/15/86 ELEV.:

Blow Counts*	Percent Passing No. 200 Sieve	MOISTURE CONTENT % OF DRY WT.	DRY DENSITY (PCF)	DEPTH (Ft.)
28	3	13.0	98	5
49		6.9	106	10
59	5	9.2	114	15
31				20
				25
				30
				40

SAMPLE LOCATION



3" Asphalt Concrete on 5" Aggregate Base

DARK BROWN SLIGHTLY SILTY GRAVELLY SAND (SP); loose to medium dense, slightly moist; FILL.

TAN VERY COARSE GRAVELLY SAND (SP); medium dense slightly moist; FILL.

ORANGE-BROWN SANDY GRAVEL (GW); dense, dry to slightly moist, well graded; 3" maximum gravel diameter.

Water Level 7/15/86

GREY-BROWN MEDIUM TO COARSE SAND (SW).

GREY-BROWN VERY COURSE SAND (SW); dense, saturated.

J.H. KLEINFELDER & ASSOCIATES GEOTECHNICAL CONSULTANTS • TESTING LABORATORIES	LOG OF BORING 2	PLATE
FILE NO: R-1665-1 DRAWN BY: DATE: 8-13-86 CHKD. BY: [illegible]	PROPOSED SQUAW VALLEY INN EXPANSION SQUAW VALLEY, CALIFORNIA	3

LOG OF Boring 3

EQUIPMENT: CME 55 Auger Drill Rig

DATE: 7/15/86 **ELEV.:**

Blow Counts*	Percent Passing No. 200 Sieve	MOISTURE CONTENT % OF DRY WT.	DRY DENSITY (PCF)	DEPTH (Ft.)	SAMPLE LOCATION	
						DARK BROWN GRAVELLY SLIGHTLY SILTY SAND (SP); loose to medium dense, dry to slightly moist; FILL.
30						ORANGE-BROWN MEDIUM TO COARSE SAND (SP); medium dense to dense, slightly moist, some small gravel to 3/4"; FILL
				5		Becoming loose and more coarse below 4'.
7						Moist at 6-1/2'.
						ORANGE-BROWN VERY SANDY GRAVEL (GW/SW); dense, moist.
				10		
43						
40						ORANGE-BROWN GRAVELLY SAND (SP); dense, very moist to wet.
				15		Water Level 7/15/86.
36						ORANGE-BROWN VERY SANC GRAVEL (GW/SW); dense, saturated.
						DARK BROWN FINE TO COARSE SAND (SW/SP); medium dense, saturated.
				20		
16						Very fine grained and loose below 24'.
				25		
5						GRAY HIGHLY PLASTIC SILT (MH); medium, stiff, saturated.
						RED-BROWN VERY COARSE GRAVELLY SAND (SP); dense, saturated.
				30		
37						
				40		

J.H. KLEINFELDER & ASSOCIATES
GEOTECHNICAL CONSULTANTS • TESTING LABORATORIES

FILE NO: R-1665-1 DRAWN BY:
DATE: 8-13-86 CHKD. BY: [illegible]

LOG OF BORING 3

PROPOSED SQUAW VALLEY INN
EXPANSION
SQUAW VALLEY, CALIFORNIA

PLATE 4

LOG OF Boring 3

EQUIPMENT: CME 55 Auger Drill Rig

DATE: 7/15/86 **ELEV.:**

Blow Counts*	Percent Passing No. 200 Sieve	MOISTURE CONTENT % OF DRY WT.	DRY DENSITY (PCF)	DEPTH (Ft.)	SAMPLE LOCATION		
67							MEDIUM TO RED-BROWN SANDY GRAVEL (GP/SP); very dense, saturated.
				45			

J.H. KLEINFELDER & ASSOCIATES
GEOTECHNICAL CONSULTANTS • TESTING LABORATORIES

FILE NO: R-1665-1 DRAWN BY:

DATE: 8 13 86 CHKD. BY: [illegible]

LOG OF BORING 3

PROPOSED SQUAW VALLEY INN EXPANSION
SQUAW VALLEY, CALIFORNIA

PLATE 5

LOG OF Boring 4

EQUIPMENT: CME 55 Auger Drill Rig

DATE: 7/15/86 **ELEV.:**

Blow Counts*	Percent Passing No. 200 Sieve	MOISTURE CONTENT % OF DRY WT.	DRY DENSITY (PCF)	DEPTH (Ft.)	SAMPLE LOCATION	
14	13					DARK BROWN GRAVELLY SILTY SAND (SM); medium dense, slightly moist; FILL.
						ORANGE-BROWN COARSE SAND (SW); medium dense, dry to slightly moist; FILL.
				5		BROWN VERY GRAVELLY SAND (SP/GP); very dense moist.
67		5.3	114			
				10		
4 8	43	38.8	82			DARK BROWN VERY CLAYEY SAND (SC/CL); loose, very moist to wet.
						ORANGE BROWN GRAVELLY SAND (SP); very moist to wet, 2" maximum gravel size.
				15		
27						
				20		No Free Water Encountered
				25		
				30		
				40		

J.H. KLEINFELDER & ASSOCIATES
GEOTECHNICAL CONSULTANTS • TESTING LABORATORIES

FILE NO: R-1665-1 DRAWN BY:

DATE: 8-13-86 CHKD. BY: [illegible]

LOG OF BORING 4

PROPOSED SQUAW VALLEY INN EXPANSION
SQUAW VALLEY, CALIFORNIA

PLATE 6

LOG OF Boring 5

EQUIPMENT: CME 55 Auger Drill Rig

DATE: 7/15/86 **ELEV.:**

Blow Counts*	Percent Passing No. 200 Sieve	MOISTURE CONTENT % OF DRY WT.	DRY DENSITY (PCF)	DEPTH (Ft.)
9		7.4	96	
				5
26				
				10
11	3	26.5	91	
				15
3	46	34.2	87	
				20
22				
				25
15		41.1	79	
				30
41				
44				
95				
				40

DARK BROWN SILTY GRAVELLY SAND (SP/SM); loose, slightly moist, 2-1/2" maximum gravel size; FILL.

ORANGE-BROWN GRAVELLY COARSE SAND (SP); medium dense, slightly mcist, 3" maximum gravel size.

BROWN FINE SAND (SW); loose to medium dense, moist, color change at 11' to orange brown, and coarse grained sand.

DARK BROWN VERY FINE VERY SILTY SAND (SM/ML); loose, wet to saturated.

Water Level 7/16/86.

BROWN SANDY, GRAVEL (GP); medium dense to dense, saturated.

BROWN FINE SAND (SP); medium dense, saturated.

ORANGE BROWN VERY SANDY GRAVEL (SP/SP); medium dense, saturated.

MEDIUM BROWN VERY FINE SLIGHTLY PLASTIC CLAYEY SAND (SC/CL); loose, saturated.

ORANGE-BROWN CLAYEY GRAVELLY SAND (SP/SC); medium dense to dense, saturated, clay decreased in depth.

BROWN TO ORANGE BROWN GRAVELLY SAND (SP); dense to very dense, saturated; with very fine sand interbeds.

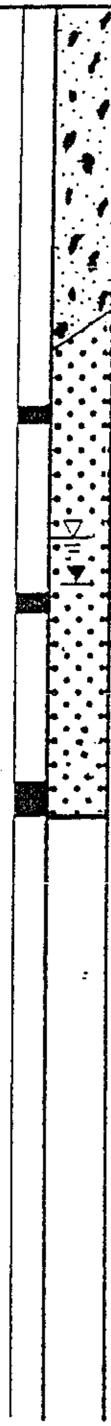
J.H. KLEINFELDER & ASSOCIATES GEOTECHNICAL CONSULTANTS • TESTING LABORATORIES	LOG OF BORING 5	PLATE
FILE NO: R-1665-1 DRAWN BY: DATE: 8-3-86 CHKD. BY: [illegible]	PROPOSED SQUAW VALLEY INN EXPANSION SQUAW VALLEY, CALIFORNIA	7

LOG OF Boring 6

EQUIPMENT: CME 55 Auger Drill Rig

DATE: 7/15/86 **ELEV.:**

Blow Counts*	Percent Passing No. 200 Sieve	MOISTURE CONTENT % OF DRY WT.	DRY DENSITY (PCF)	DEPTH (Ft.)	SAMPLE LOCATION	Description
						BROWN SILTY VERY SANDY GRAVEL (GW/SW); loose in upper 2', medium dense to very dense, slightly moist to moist below 2', 2-1/2" maximum gravel size.
49						
				5		
						ORANGE-BROWN GRAVELLY SAND (SP); medium dense to very dense, very moist to saturated.
				10		
38						
						Water Level 7/16/86 (Approximate)
						Water Level 7/18/86 (Stabilized)
				15		
32						
				20		
61						
				25		
				30		
				40		



J.H. KLEINFELDER & ASSOCIATES GEOTECHNICAL CONSULTANTS • TESTING LABORATORIES	LOG OF BORING 6	PLATE
FILE NO: R-1665-1 DRAWN BY: DATE: 8-13-86 CHKD. BY: CJ	PROPOSED SQUAW VALLEY INN EXPANSION SQUAW VALLEY, CALIFORNIA	8

LOG OF Boring 7

EQUIPMENT: CME 55 Auger Drill Rig

DATE: 7/15/86 ELEV.:

Blow Count*	Percent Passing No. 200 Sieve	MOISTURE CONTENT % OF DRY WT.	DRY DENSITY (PCF)	DEPTH (Ft.)	SAMPLE LOCATION	
						2-1/2" AC (No aggregate base).
20	9					BROWN SILTY FINE TO COARSE SAND (SP/SM); medium dense, slightly moist; FILL.
				5		
				10		
				15		

LOG OF

EQUIPMENT:

DATE: ELEV.:

Blow Count*	Percent Passing No. 200 Sieve	MOISTURE CONTENT % OF DRY WT.	DRY DENSITY (PCF)	DEPTH (Ft.)	SAMPLE LOCATION	
				5		
				10		
				15		

J.H. KLEINFELDER & ASSOCIATES
GEOTECHNICAL CONSULTANTS • TESTING LABORATORIES

FILE NO: R-1665-1 DRAWN BY:

DATE: 8-13-86 CHKD. BY: [illegible]

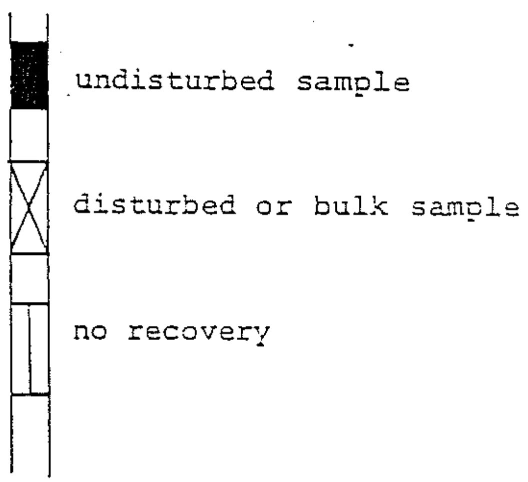
LOG OF BORING 7

PROPOSED SQUAW VALLEY INN
EXPANSION
SQUAW VALLEY, CALIFORNIA

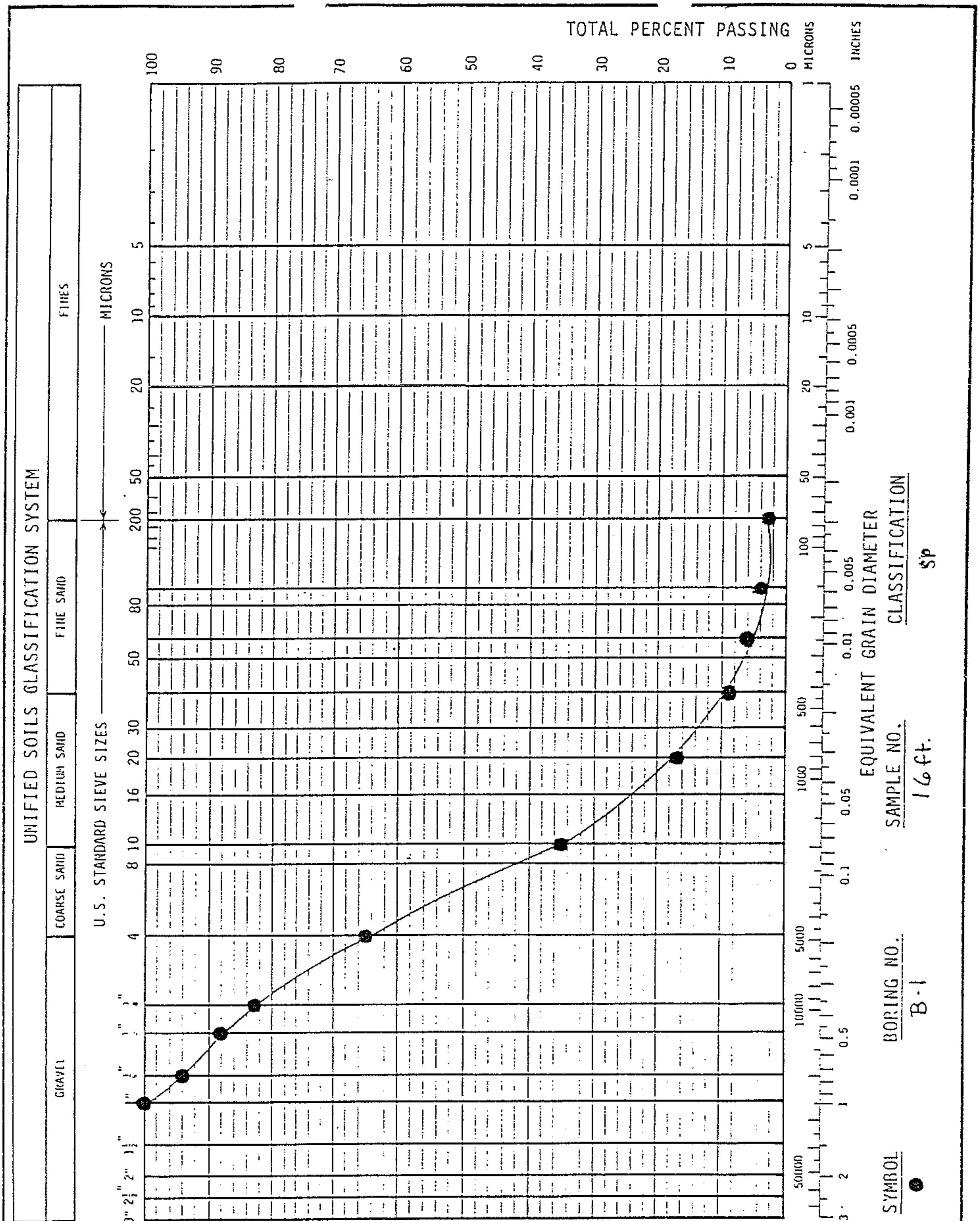
PLATE 9

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS MORE THAN HALF IS LARGER THAN #200 SIEVE	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL GRADED GRAVELS, GRAVEL - SAND MIXTURES
			GP	POORLY GRADED GRAVELS, GRAVEL - SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, POORLY GRADED GRAVEL - SAND - SILT MIXTURES
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL - SAND - CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS
			SP	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, POORLY GRADED SAND - SILT MIXTURES
			SC	CLAYEY SANDS, POORLY GRADED SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN HALF IS SMALLER THAN #200 SIEVE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt	PEAT AND OTHER HIGHLY ORGANIC SOILS
FILL MATERIAL				AS DESCRIBED ON LOG

UNIFIED SOIL CLASSIFICATION SYSTEM



J.H. KLEINFELDER & ASSOCIATES GEOTECHNICAL CONSULTANTS • TESTING LABORATORIES	KEY TO BORING LOGS	PLATE 10
FILE NO: ______ DRAWN BY: ______ DATE: 8/3/82 CHKD. BY: [illegible]	PROPOSED SQUAW VALLEY INN EXPANSION SQUAW VALLEY, CALIFORNIA	



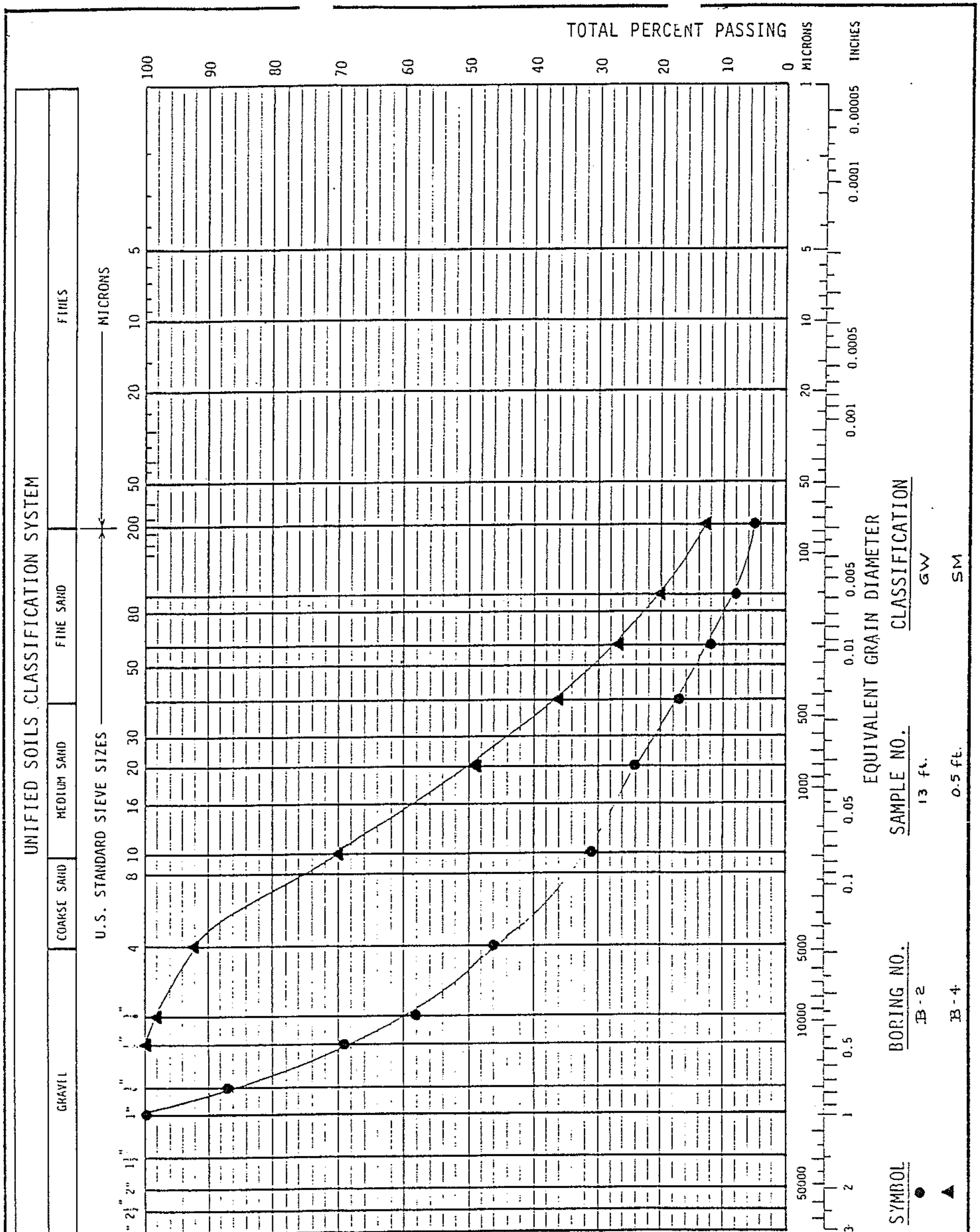
J.H. KLEINFELDER & ASSOCIATES
GEOTECHNICAL CONSULTANTS • MATERIALS TESTING

PROPOSED SQUAW VALLEY INN
EXPANSION
SQUAW VALLEY, CALIFORNIA

GRAIN SIZE DISTRIBUTION

PLATE
11

PROJECT NO. R-1665-1



J.H. KLEINFELDER & ASSOCIATES

GEOTECHNICAL CONSULTANTS • MATERIALS TESTING

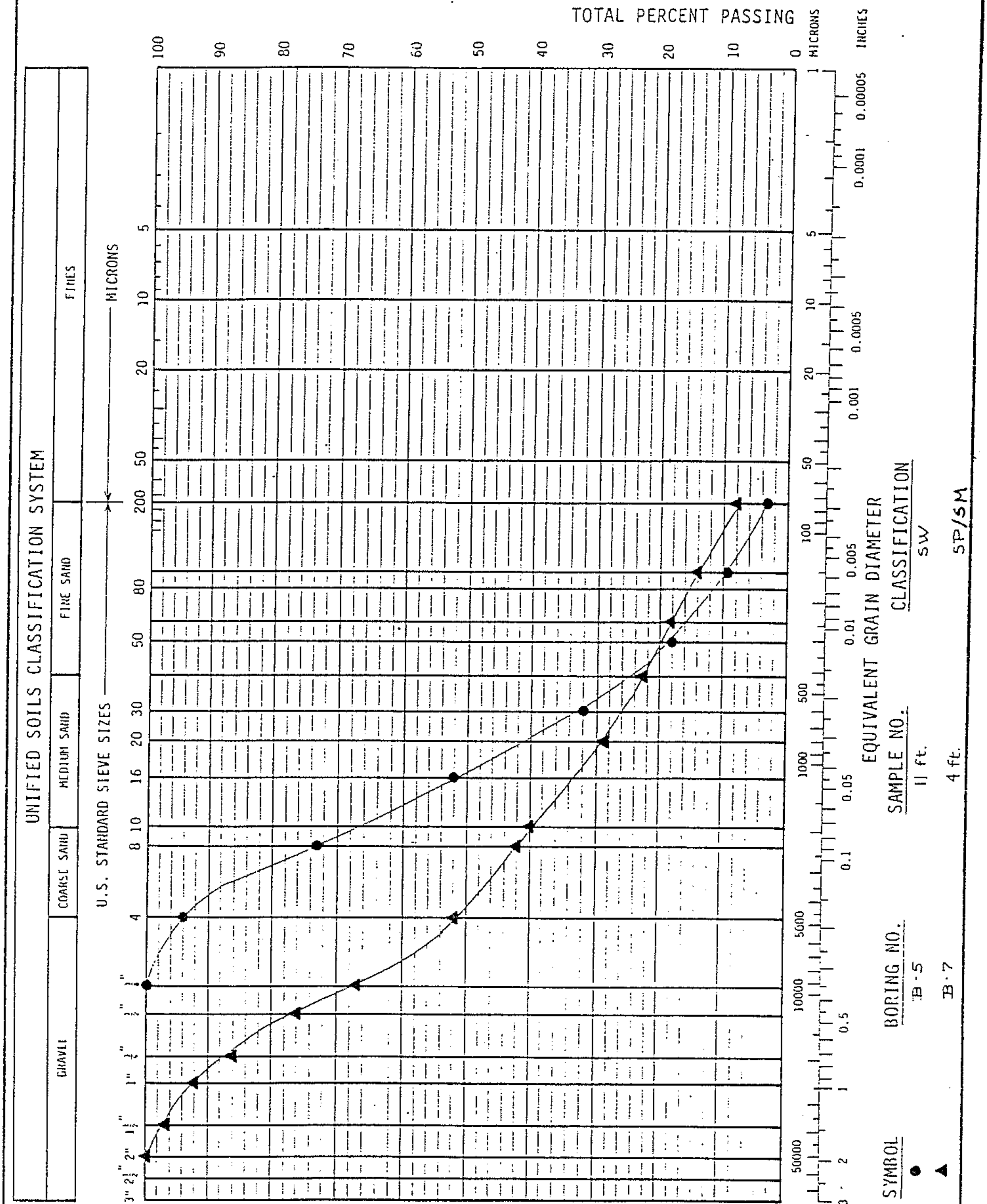
PROPOSED SQUAW VALLEY INN EXPANSION
SQUAW VALLEY, CALIFORNIA

GRAIN SIZE DISTRIBUTION

PLATE

12

PROJECT NO. P-1665-1



J.H. KLEINFELDER & ASSOCIATES

GEOTECHNICAL CONSULTANTS • MATERIALS TESTING

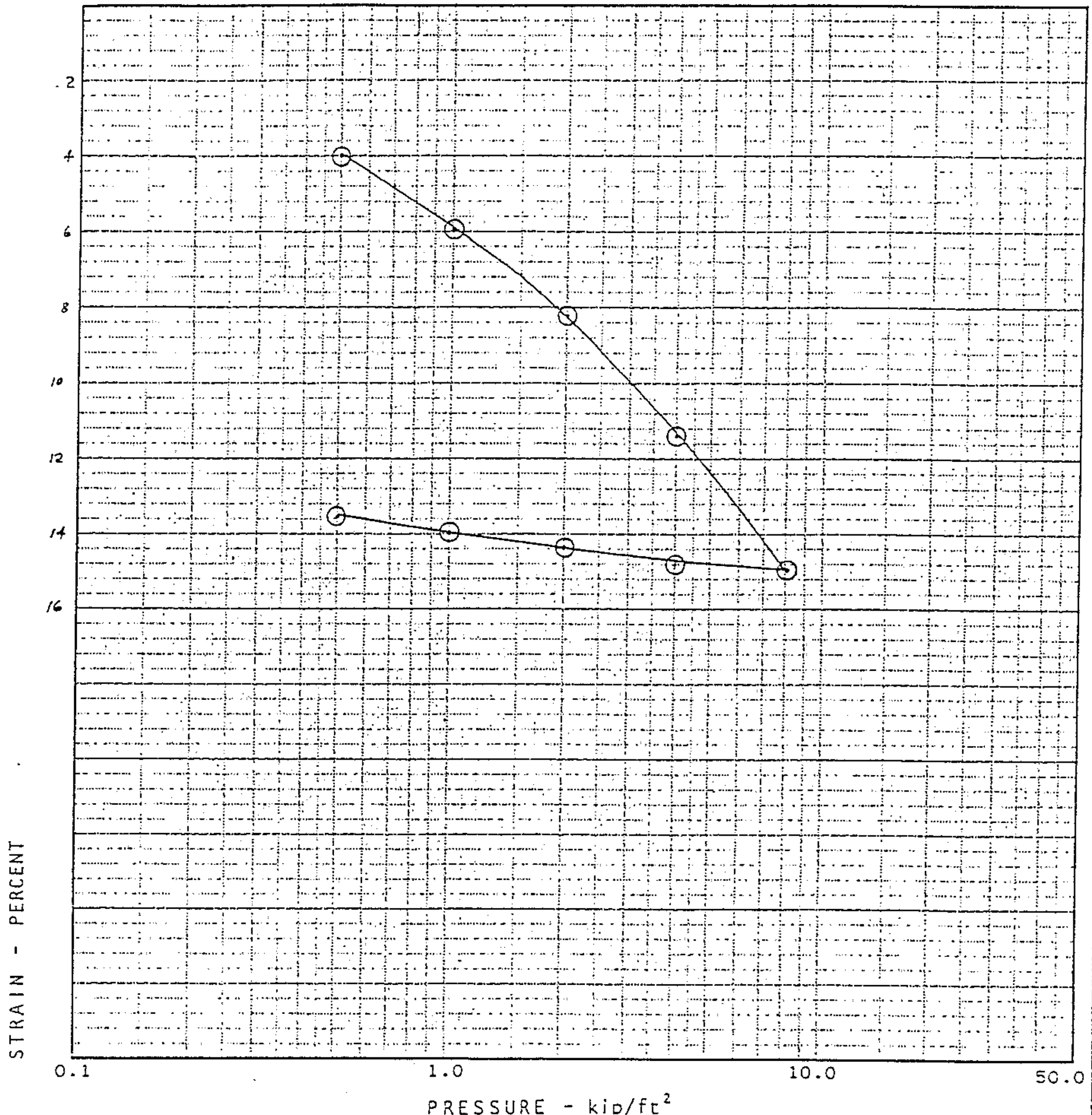
PROJECT NO. R-1665-1

PROPOSED SQUAW VALLEY INN
EXPANSION
SQUAW VALLEY, CALIFORNIA

GRAIN SIZE DISTRIBUTION

PLATE

13



BORING NO. B-5
DEPTH 26'
SAMPLE DESCRIPTION Brown very clayey very fine sand (SC/CL)
OVERBURDEN PRESSURE 2300 psf (Approx.)
PRECONSOLIDATION PRESSURE 2200 psf (Apx.)
COMPRESSION INDEX, C_c 13.7% per ksf
RECOMPRESSION INDEX, C_r 1.2% per ksf

	INITIAL	FINAL
DRY DENSITY - lb/ft³	78.8	91.3
WATER CONTENT - %	41.1	33.2
VOID RATIO		
DEGREE OF SATURATION, %		
SAMPLE HEIGHT - inches	1.0000	0.8623

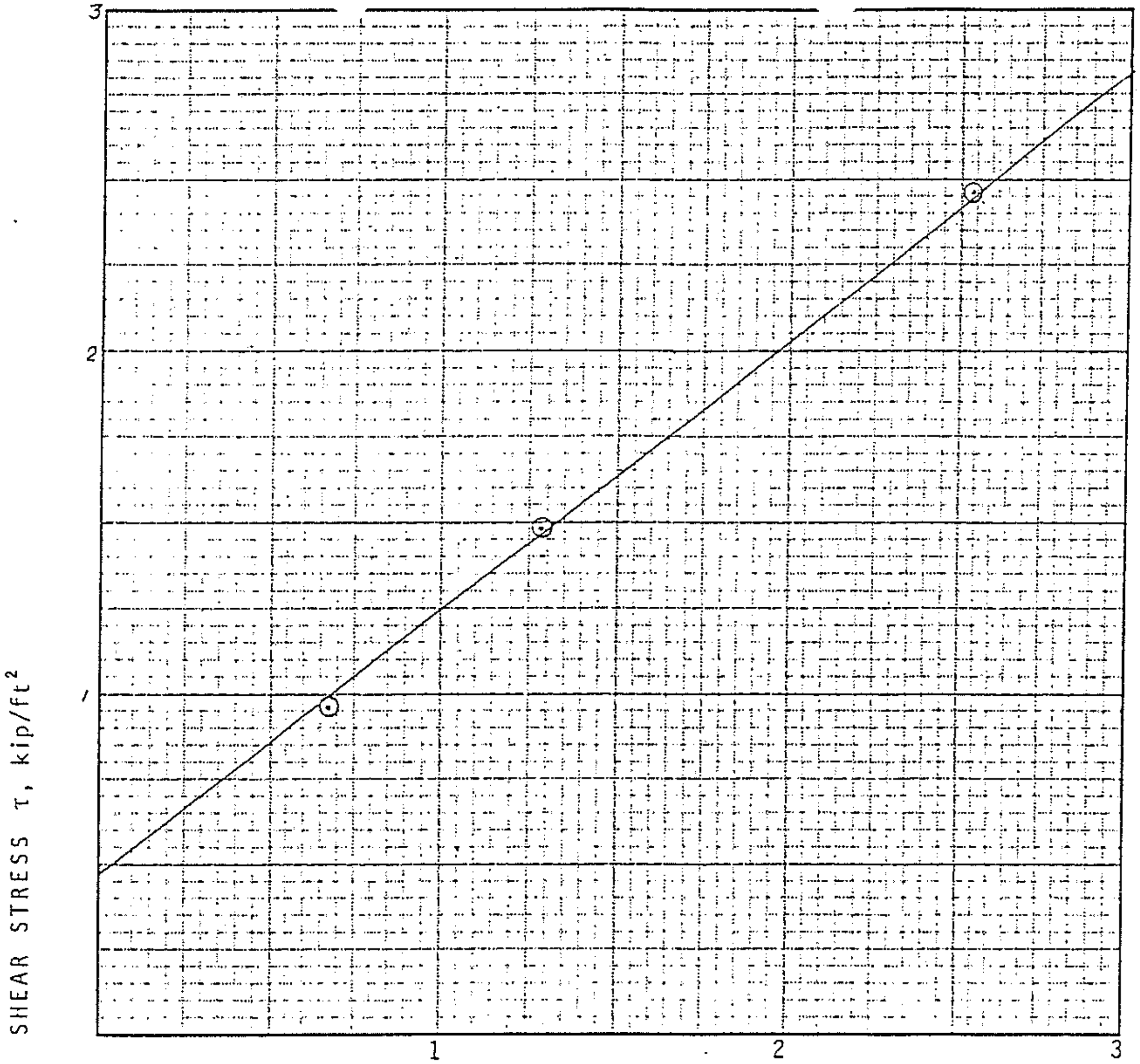
J.H. KLEINFELDER & ASSOCIATES
GEOTECHNICAL CONSULTANTS • MATERIALS TESTING

PROJECT NO. R-1665-1

PROPOSED SQUAW VALLEY INN
EXPANSION
SQUAW VALLEY, CALIFORNIA

CONSOLIDATION TEST

PLATE 14



NORMAL STRESS σ, kip/ft²

BORING NO. B-2 SAMPLE NO. B-2 @ 3.5 - 4' DEPTH, ft 3.5 - 4'

DESCRIPTION Brown coarse sand with gravel and rock.

SYMBOL			
DRY DENSITY lb/ft³	98	98	98
INITIAL WATER CONTENT %	13.0	13.0	13.0
FINAL WATER CONTENT %	14.6	14.6	14.6
NORMAL STRESS σ, kip/ft²	0.669	1.287	2.545
SHEAR STRESS τ, kip/ft²	0.961	1.490	2.461

ANGLE OF INTERNAL FRICTION, ϕ 38°

COHESION, kip/ft² 0.50

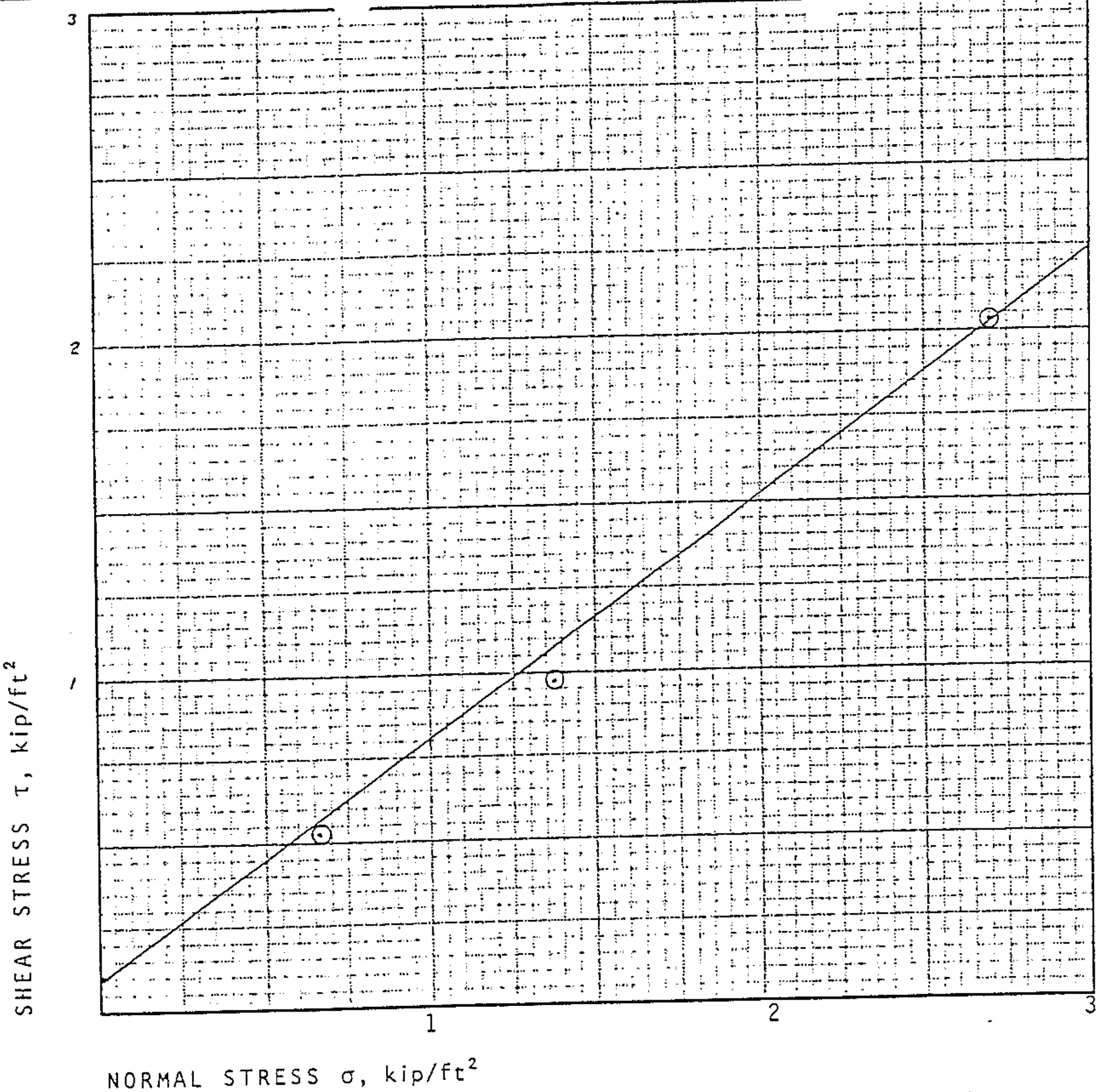
J.H. KLEINFELDER & ASSOCIATES
GEOTECHNICAL CONSULTANTS • MATERIALS TESTING

PROPOSED SQUAW VALLEY INN
EXPANSION
SQUAW VALLEY, CALIFORNIA

DIRECT SHEAR TEST

PLATE 15

PROJECT NO. R-1665-1



BORING NO. B-5 SAMPLE NO. B-5 @ 11' DEPTH, ft 11'

DESCRIPTION Orange coarse silty sand.

SYMBOL			
DRY DENSITY lb/ft^3	91	91	91
INITIAL WATER CONTENT %	26.4	26.4	26.4
FINAL WATER CONTENT %	24.8	24.8	24.8
NORMAL STRESS σ, kip/ft^2	0.676	1.375	2.700
SHEAR STRESS τ, kip/ft^2	0.512	0.979	2.063

ANGLE OF INTERNAL FRICTION, ϕ 35°

COHESION, kip/ft^2 0.10

J.H. KLEINFELDER & ASSOCIATES
GEOTECHNICAL CONSULTANTS • MATERIALS TESTING

PROPOSED SQUAW VALLEY INN EXPANSION
SQUAW VALLEY, CALIFORNIA

DIRECT SHEAR TEST

PLATE 16

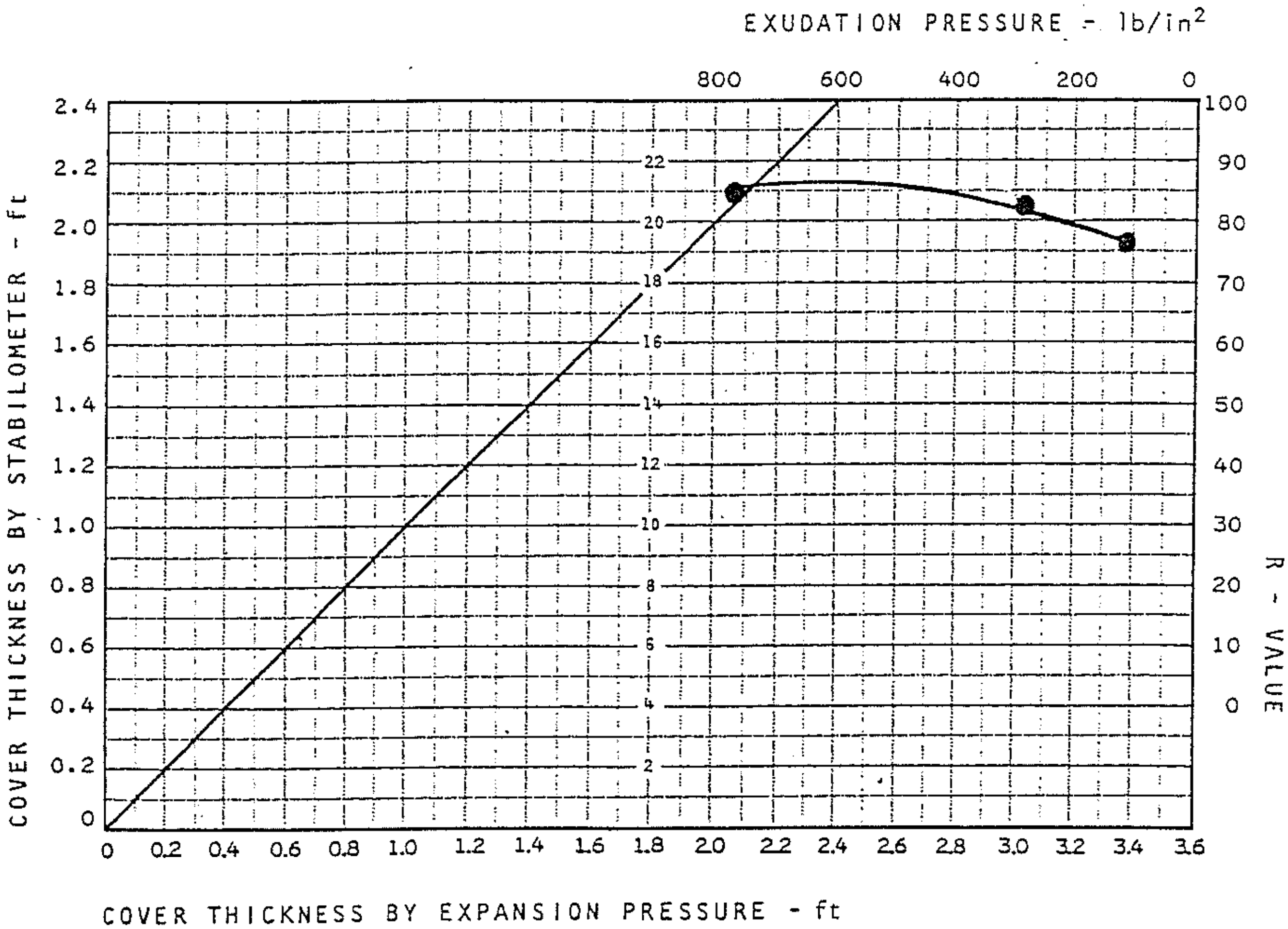
PROJECT NO. R-1665-1

JHK & A Form R-8

SAMPLE LOCATION: B-4 @ 0.5'

SAMPLE DESCRIPTION: Dark brown sandy silt with gravels

DATE SAMPLED: 7/21/86



SPECIMEN	A	B	C
EXUDATION PRESSURE, lb/in^2	760	292	104
EXPANSION DIAL (.0001")	15	8	0
EXPANSION PRESSURE, lb/ft^2	65	35	0
RESISTANCE VALUE, R	84	83	76
% MOISTURE AT TEST	9.8	11.0	12.2
DRY DENSITY AT TEST, lb/ft^3	124.5	121.2	119.0
R VALUE AT 300 lb/in^2 EXUDATION PRESSURE	83		
R VALUE BY EXPANSION PRESSURE (TI =)			

J.H. KLEINFELDER & ASSOCIATES
GEOTECHNICAL CONSULTANTS • MATERIALS TESTING

PROPOSED SQUAW VALLEY INN
EXPANSION
SQUAW VALLEY, CALIFORNIA

RESISTANCE VALUE

PLATE **17**

PROJECT NO. R-1665-1

APPENDIX A

SUGGESTED SPECIFICATIONS FOR EARTHWORK AND PAVEMENT CONSTRUCTION FOR THE PROPOSED SQUAW VALLEY INN PROJECT SQUAW VALLEY, CALIFORNIA

1.0 GENERAL

1.1 Scope - The work done under these specifications shall include clearing, stripping, removal of unsuitable material, excavation, installation of subsurface drainage, preparation of natural soils, placement and compaction of on-site and imported fill material and placement and compaction of pavement materials.

1.2 Contractor's Responsibility - A preliminary soils investigation was performed for the project by J. H. Kleinfelder & Associates dated August 14, 1986. The Contractor shall attentively examine the site in such a manner that he can confirm existing surface conditions with those presented in the soils report. He shall satisfy himself that the quality and quantity of exposed materials and subsurface soil or rock deposits have been satisfactorily represented by the Soils Engineer's report and Civil Engineer's drawings. Any discrepancy that may be of prior knowledge to the Contractor or that is revealed through his investigations shall be made available to the Owner. It is the Contractor's responsibility to review the attached report prior to construction. The selection of equipment for use on the project and the order of work will similarly be his responsibility such that the requirements included in following sections have been met.

1.3 Soils Engineer - The work covered by these specifications shall be observed and tested by the Soils Engineer, J. H. Kleinfelder & Associates, who shall be hired by the Owner. The Soils Engineer will be present during the site preparation and grading to observe the work and to perform the tests necessary to evaluate material quality and compaction. The Soils Engineer shall submit a report to the Owner, including a tabulation of all tests performed. The costs of retesting of unsuitable work performed by the Contractor shall be deducted from the payments to the Contractor.

1.4 Standard Specifications - Where referred to in these specifications, "Standard Specifications" shall mean the current State of California Department of Transportation (CALTRANS) Standard Specifications dated January 1981.

1.5 Compaction Test Method

1.5.1 Outside State Right-of-Ways - Where referred to herein, relative compaction outside of the state rights-of-way shall mean the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material, as determined by ASTM D1557-78 Compaction Test Procedure. Optimum moisture content shall mean the moisture content of maximum dry density as determined above.

1.5.2 Within State Right-of-Ways - The relative compaction within state right-of-ways shall mean the in-place wet density of soil expressed as a percentage of the maximum wet density of the same material, as determined by Cal Method 216. Optimum moisture content shall mean the moisture content at the maximum wet density.

2.0 SITE PREPARATION

2.1 Clearing - Areas to be graded shall be cleared and grubbed of all vegetation and debris. These materials shall be removed from the site by the Contractor.

2.2 Stripping - Surface soils containing roots and organic matter shall be stripped from areas to be graded and stockpiled or discarded as directed by the Owner. In general, the depth of stripping of the topsoil will be approximately 2 to 4 inches. Deeper stripping, where required to remove weak soils or accumulations of organic matter, shall be performed when determined by the

Soils Engineer. Strippings shall be removed from the site or stockpiled at a location designated by the Owner.

2.3 Removal of Existing Fill - Existing fill soils, trash and debris in the areas to be graded shall be removed prior to the placing of any compacted fill. Portions of any existing fills that are suitable for use in compacted fill may be stockpiled for future use. All organic material, topsoil, expansive soils, oversize material or other unsuitable material shall be removed from the site by the Contractor or disposed of at a location on-site, if so designated by the Owner.

2.4 Ground Surface - The ground surface exposed by stripping shall be scarified to a depth of 6 inches, moisture conditioned to the proper moisture content for compaction, and compacted to a minimum of 90% relative compaction. Recompaction shall be approved by the Soils Engineer prior to placing fill.

3.0 EXCAVATION

3.1 General - Excavations shall be performed to the lines and grades indicated on the plans.

The data presented in the soils report is for information only and the Contractor shall make his own interpretation with regard to the methods and equipment necessary to perform the excavation and to obtain material suitable for fill.

3.2 Materials - Soils which are removed and are unsuitable for fill should be placed in non-structural areas of the project. Where necessary, these soils may be placed in deeper fills if approved by the Soils Engineer.

All oversized rocks and boulders that cannot be incorporated in the work by placing in embankments or used as rip-rap or for other purposes shall be removed from the site by the Contractor.

3.3 Treatment of Exposed Surface - The ground surface exposed by excavation shall be scarified to a depth of 6 inches, moisture conditioned to the proper moisture content for compaction, and compacted as required for compacted fill. Recompaction shall be approved by the Soils Engineer prior to placing fill.

3.4 Rock Excavation - Where solid rock is encountered in excavation it shall be loosened and broken up so that no solid ribs, projections, or large fragments will be within six (6) inches of the surface of the final subgrade.

4.0 COMPACTED FILL

4.1 Materials - Fill material shall consist of suitable on-site or imported fill. All materials used for structural fill shall be reasonably free of organic material, have a liquid limit less than 25, a plasticity index less than 12, 100% passing the 6 inch sieve and less than 25 passing the #200 sieve.

4.2 Placement - All fill materials shall be placed in layers of 8 inches or less in loose thickness and uniformly moisture conditioned. The lift should then be compacted with a sheepsfoot roller or other approved compaction equipment to achieve at least 90% relative compaction in areas under structures, and to at least 85% in undeveloped areas. A relative compaction of at least 90 percent should be achieved in utility trench backfill and under pavements on private property. A relative compaction of at least 95 percent is required for a depth of 2.5 feet below finish grade in all fill areas within the state right-of-ways. No fill material shall be placed, spread or rolled while it is frozen or thawing, or during unfavorable weather conditions.

4.3 Benching - Fill placed on slopes steeper than 5 horizontal to 1 vertical shall be keyed into firm, native soils or rock by a series of benches. Benching can be conducted simultaneously with placement of fill. However, the method and extend of benching shall be checked by the Soils Engineer.

4.4 Compaction Equipment - The Contractor shall provide and use sufficient equipment of a type and weight suitable for the conditions encountered in the field. The equipment shall be capable of obtaining the required compaction in all areas, including those that are inaccessible to ordinary rolling equipment.

4.5 Recompaction - When, in the judgement of the Soils Engineer, sufficient compaction effort has not been used, or where the field density tests indicate that the required compaction or moisture content has not been obtained, or if "pumping" or other indications of instability are noted, the fill shall be reworked and recompacted as needed to obtain a stable fill at the required density and moisture content prior to placing additional fill materials.

4.6 Responsibility - The Contractor shall be responsible for the maintenance and protection of all embankments and fills made during the contract period and shall bear the expense of replacing any portion which has become displaced due to carelessness, negligent work or failure to take proper precautions.

5.0 UTILITY TRENCH BEDDING AND BACKFILL

5.1 Material - Pipe bedding shall be defined as all material within 6 inches of the perimeter of the pipe. Material for use as bedding shall consist of clean, granular material free of clay and organic material and be such a size that 90 to 100 percent will pass a No. 4 sieve and not more than 5 percent will pass a No. 200 sieve.

Backfill should be classified as all material within the remainder of the trench. Backfill shall conform to the following gradation:

Percent Passing	Sieve Size
100	3 - Inch
35 - 100	No. 4
20 - 100	No. 30

5.2 Placement and Compaction - Pipe bedding shall be placed in thin layers not exceeding eight inches in loose thickness, conditioned to the proper moisture content for compaction, and compacted to at least 90% relative compaction. All other trench backfill shall be placed in thin layers not exceeding 8 inches in loose thickness, conditioned to the proper moisture content, and compacted as required for adjacent fill. If not specified, backfill should be compacted to at least 90% relative compaction in areas under structures, utilities, roadways, parking areas and concrete flatwork and to 85% relative compaction in undeveloped areas. The relative compaction in the upper 2.5' below finish grade within state right-of-ways shall be a minimum of 95 percent.

6.0 SUBSURFACE DRAINAGE

6.1 General - Subsurface drainage should be constructed as shown on the plans. Drainage pipe should meet the requirements set forth in the Standard Specifications.

6.2 Materials - Permeable "drain rock" material used for subdrainage should meet the following gradation requirements or consist of other material approved by the Soils Engineer.

Sieve Size	Percentage Passing
2"	100
1½"	95 - 100
3/4"	50 - 100
3/8"	15 - 55
No. 4	0 - 25
No. 100	0 - 5
No. 200	0 - 3

6.3 Geotextile Fabric - Nonwoven filter fabric should be placed between the permeable drain rock and native soils. Filter cloth with an equivalent opening size greater than the No. 100 sieve size, and a grab strength not less 100 lbs should be used. We should be consulted on a specific basis when a particular fabric is chosen so that compliance to the above recommendations can be verified.

6.4 Placement & Compaction - Drain rock shall be placed in thin layers not exceeding 8 inches in loose thickness and compacted as required for adjacent fill, but in no case will be less than 85% relative compaction. Placement of geotextile fabric will be in accordance with the manufacturer's specifications, and should be checked by the Soils Engineer.

7.0 AGGREGATE BASE FOR CONCRETE SLABS

7.1 Material - Aggregate base for concrete slabs shall consist of clean free-draining sand, gravel or crushed rock conforming to the following gradation:

Sieve Size	Percent Passing
1"	100
3/8"	30 - 100
No. 200	0 - 10

7.2 Placement - Aggregate base shall be compacted and kept moist until placement of concrete. Compaction shall be by suitable vibrating compactors. Aggregate base shall be placed in layers not exceeding eight inches in thickness. Each layer shall be compacted by at least four passes of the compaction equipment or until 95% relative compaction has been obtained.

8.0 SUBGRADE AND AGGREGATE BASE FOR PAVED AREAS

8.1 Subgrade Preparation - After completion of the utility trench backfill and prior to placement of aggregate base, the upper six inches of subgrade soil shall be uniformly compacted to at least 95 percent relative compaction. This may require scarifying, moisture conditioning, and compacting in both cut and fill areas.

8.2 Aggregate Base - Aggregate materials shall meet the requirements of the appropriate sections of the "Standard Specifications" for Class 2 Aggregate Base. The aggregate base materials must be approved by the Soils Engineer prior to use.

After the subgrade is properly prepared, the aggregate base shall be placed in layers, moisture conditioned as necessary, and compacted by rolling to at least 95% relative compaction. The compacted thickness of aggregate base shall be shown on the approved plans.

9.0 ASPHALT CONCRETE PAVEMENT

9.1 Thickness - The compacted thickness of asphalt concrete shall be as shown on the approved plans.

9.2 Materials - Aggregate materials for asphalt concrete shall conform to the requirements listed for Type A or Type B Bituminous Aggregates in Section 39 of the "Standard Specifications" and utilize AR-4000 grade of asphalt concrete. The Contractor shall submit a proposed asphalt concrete mix design to the Owner for review and approval prior to paving. The mix design shall be based on the Hveem Method.

Where prime coat is specified, the type and grade of asphalt for use as prime coat shall be MC250 or RC250 with an application rate of 0.20 to 0.30 gallons per square yard. The type and grade of asphalt for use as tack coat shall be SS1 or SS1h with an application rate of 0.10 to 0.15 gallons per square yard.

The type and grade of asphalt for use as seal coat shall be MC250 or RC250 with an application rate of 0.15 to 0.20 gallons per square yard. Sand blotter, if needed to prevent "pick-up", shall be spread at a rate of 10 to 15 pounds per square yard.

9.3 Placement - The asphalt concrete material and placement procedures shall conform to appropriate sections of the "Standard Specifications".